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Chemical Treatment

of

Combined Sewer Overflows

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Less than 8% of the total sanitary sewage flow collected in the Milk River combined sewer has been by-passed during periods of storm overflow during the past 8 years. Total overflow volumes did not exceed 5.5 million cubic ft. 90% of the time. Initial BOD over 18 samples ranged from 23 mg/l to 376 mg/l with an average of 110 mg/l, while suspended solids averaged 249 mg/l. On final samples taken during the same period, BOD values averaged 42 mg/l and SS values averaged 67 m/gl. Chlorine demand for 56 influent samples averaged 7.7 mg/l. The receiving waters for these overflows were characterized by visual observation as being highly polluted. Cationic polymeric flocculants and flocculent aids were tested on combined overflows from the sewer system. It was found that the existing Milk River pumping station and retention basin were suitable for modification, namely the addition of weirs and variable rate pumping, to acheive an average SS removal of 63%. tThe installation of disinfection equipment was estimated to cost $20,625 with $7,700 annually for chemical costs. This equipment would produce a coliform density of 1000/100 ml at an average cost of approximately $.012/1000 gallons. (Lowry-Texas)

(WRSC abstract)

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CHEMICAL TREATMENT OF COMBINED SEWER OVERFLOWS

Study of Flocculant Treatment and Disinfection of Milk River Pumping Station Combined Sewer Overflows at Grosse Pointe Woods, Michigan

Environmental Protection Agency
Water Quality Office

by

The Dow Chemical Company
Midland, Michigan

Contract No. 14-12-9
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September 1970

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ABSTRACT

Overflows of sanitary sewage with stormwater contribute to pollution and deterioration of water resources. The description of a typical pumping station, the characteristics of combined sewage overflows, and chemical treatment of overflow with chemical flocculants and disinfectants are included herein.

Combined sewage from a residential community of 3990 acres in northeast Detroit, Michigan, is collected at the Milk River Pumping Station. During eight years of record, overflow rates did not exceed 410 cfs (184,000 gpm) for 90 percent of the pumping time. Overflow volumes did not exceed 5.5 million cubic feet (41.1 million gallons) for 90 percent of the occurrences. The average number of days of pumping per year (41) is about equal to the average number of days per year having precipitation >0.2 inches (45).

Twenty-two analyses were performed on consecutive time-weighted samples of influent and effluent during thirty-three storms over a two-year period. Biochemical oxygen demand and suspended solids decreased after initial flushing of the sewers. Chlorine demand values remained relatively constant throughout storms. The Milk River channel and the immediate receiving bay were severely polluted.

Cationic polymeric flocculants and flocculant aids significantly improved removal of suspended solids from combined sewage as demonstrated by tests in beakers and a long-tube sedimentation column. Adequate disinfection of the combined sewage before discharge to the channel is possible. The performance of the existing basin can be improved by the use of staged continuous pumping at lower rates and the addition of baffles for improved flow distribution.

This report was submitted in fulfillment of Contract 14-12-9 between the Environmental Protection Agency, Water Quality Office, and The Dow Chemical Company.

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SECTION 1

CONCLUSIONS

BACKGROUND DATA AND ANALYSIS (5)

1. During eight years of record, approximately 39 percent of the total rainfall falling on the Milk River basin was collected in the Milk River Sewer System. Approximately 53 percent of this runoff was pumped to the Milk River Drain as combined sewage overflow.

2. Less than 8 percent of the total sanitary flow collected in the Milk River Combined Sewer was discharged along with storm runoff to the Milk River Drain during storm overflow pumping events.

3. During eight years of record, approximately 74 percent of the volume discharged to Lake Saint Clair at Milk River was pumped at a rate less than or equal to 410 cfs (184,000 gpm) (17 percent of 10 year design flow). Overflow rates were equal to or less than 410 cfs for over 89 percent of the pumping time.

4. During eight years of record, overflow volumes at the Milk River Pumping Station did not exceed 5.5 million cubic feet (41.1 million gallons) for 90 percent of the occurrences.

5. The average number of days of storm pumping per year (41) is approximately equal to the average number of days per year having precipitation in excess of 0.2 inches (45).

6. The initial biochemical oxygen demand of 18 combined sewage influent samples at Milk River had an average value of 110 mg/l and varied from 23 mg/l to 376 mg/l. Final samples averaged 42 mg/l and ranged from 17 mg/l to 120 mg/l.

7. Concentrations of suspended solids diminished significantly during the course of combined sewage overflows at Milk River. Initial influent samples for 18 storms averaged 249 mg/l while final samples for the same storms averaged 67 mg/l. Initial samples ranged from 766 mg/l to 32 mg/l and final samples ranged from 169 mg/l to 24 mg/l.

8. The chlorine demand of 56 combined sewage influent samples at Milk River had an average value of 7.7 mg/l and varied from a minimum of 3.5 mg/l to a maximum of 12.2 mg/l.

9. Total phosphorus concentrations in 64 Milk River influent samples varied from 0.3 mg/l to 3.7 mg/l with an average of 1.42 mg/l.

10. The Milk River channel including some of the immediate receiving bay areas of Lake Saint Clair are severely polluted as characterized by heavy sludge deposits, the existence of only highly pollution-tolerant life, high coliform counts, reduced dissolved oxygen, gas production in the channel, and other qualitative observations.

11. A transitory zone exists out into Lake Saint Clair between the severely polluted Milk River channel and the clean water areas of the Lake.

12. No measurable changes in the bottom-dwelling biota or the extent of sludge deposition could be observed in the Milk River channel or Lake Saint Clair as a result of treatment by polymeric flocculants at the Milk River Pumping Station.

TREATMENT OF COMBINED SEWAGE WITH POLYMERIC FLOCCULANTS (6)

1. Coagulant aids specifically 15 mg/l of Baroid Hectorite® clay can be utilized with the cationic flocculant systems to produce significantly improved turbidity removal from Milk River Pumping Station combined sewage.

2. Two of the cationic flocculants, PURIFLOC® C31 and PURIFLOC C32, consistently exhibited excellent flocculation activity and effective turbidity removals at concentrations of approximately 10 mg/l during the early to mid-storm periods when suspended solids loadings were relatively high.

3. During the mid- to late-storm periods, when suspended solids loadings were relatively low, PURIFLOC C31 and PURIFLOC C32 exhibited relatively poor flocculation activity.

4. Final overhead turbidity was relatively constant after treatment by PURIFLOC C31 and PURIFLOC C32 during all storm periods; final overhead quality is apparently independent of initial sample turbidity.

5. The rate of floc formation can be increased significantly by the addition of anionic polymer flocculants to the cationic flocculant systems, but may result in a corresponding decrease in solids capture efficiency.

6. The anionic and nonionic polymeric flocculants exhibited no significant flocculation activity and were not effective in reducing influent turbidity.

7. Because of the inherent settling inefficiency of the retention basin, the difficulty in controlling flocculant concentrations during short periods of pumping, and lack of a control, quantitative evaluation of the full-scale treatment studies was not possible.

8. Fair to good flocculation of combined sewage was observed in the Milk River Retention Basin during the full-scale treatment studies but sedimentation of these flows was not possible.

LONG TUBE SEDIMENTATION STUDIES (7)

1. Accurate rate data can be collected from long-tube sedimentation devices if flocculation is external to the column and sedimentation is conducted under dynamic conditions to prevent deposition.

2. Optical solids are more rapidly and more precisely determined than are gravimetric solids as indicators of sedimentation processes in small vessels.

3. Sedimentation of flocculated solids present in combined sewer overflow is Class I (discrete particles).

4. Concentration requirements of chemical flocculants can be estimated adequately on the basis of "jar" tests.

5. Efficiencies of solids removals can be estimated from initial and final depth-average solids concentrations or, preferably, extrapolated directly from reduced concentration profiles at selected overflow rates.

6. Optimized systems of flocculants were capable of removing 80 to 95 percent of optical and gravimetric solids under the idealized conditions of this study.

HYDRAULIC ANALYSIS AND MODEL STUDIES (8)

1. Effective removal of appreciable amounts of lightweight suspended solids cannot be accomplished in the existing Milk River storm overflow system because of short-circuiting, non-uniform transverse flow distribution, and inadequate retention time for plain settling.

2. Installation of variable rate pumping to provide continuous or staged operation from 0-410 cfs (0-184,000 gpm) would considerably reduce the average pumping rate for over 89 percent of the pumping time. The mean flow rate for eight years of record was estimated to be 187 cfs (84,000 gpm). The minimum sized pump at Milk River is 305 cfs (137,000 gpm).

3. Retention efficiency in the existing Milk River Basin is approximately 42 percent at a flow of 410 cfs. This corresponds to an effective retention of 8.9 minutes with a theoretical detention of 21 minutes. Optimum baffling of the basin resulted in an increase in the effective retention to approximately 20 minutes for about 96 percent efficiency.

4. Placement of some transverse weirs in the Milk River Retention Basin would significantly improve flow distribution and improve retention efficiency and solids removal, particularly at the most frequently occurring reduced flow rates.

DISINFECTION (9)

1. Differences in the killing quality of the several halogen disinfectants tested were not sufficiently pronounced in the available observations to permit conclusions on their relative merits.

2. Flocculation of combined sewage with PURIFLOC C31 was not adversely affected by disinfection with sodium hypochlorite in concentrations of up to approximately 40 mg/l.

3. The high chlorine demand of bottom sludges in the Milk River channel indicates that residual halogen would not persist in the channel under existing conditions.

4. $BrCl_5$ mixtures maintain a residual similar to Cl_2 in combined sewage samples maintained under quiescent conditions for up to 24 hours.

5. An average concentration of approximately 10 mg/l of halogen as Cl_2 appears sufficient to remove 99.99% of the total coliform in the Milk River combined sewage influent in an effective time of about 15 minutes.

ECONOMIC ANALYSES (10)

1. Where the capital cost of combined sewage treatment facilities varies approximately in proportion to plant capacity and where the occurrence of flow rates follows a log normal distribution similar to that at the Milk River Pumping Station, the cost of polymeric treatment starts to increase very rapidly at about 20 percent of the 10-year design flow of 2450 cfs (1,100,000 gpm).

2. Seventy-five percent of the flows at Milk River can be treated with NaOCl to provide an average total coliform concentration of about 1000/100 ml at a cost of approximately \$0.012/1000 gallons. Disinfection equipment installation cost would be approximately \$20,625 and chemical costs, less operation and maintenance, would be approximately \$7,700 annually.

3. The existing Milk River Pumping Station and retention basin can be modified by the addition of weirs and variable rate pumping to achieve an average suspended solids removal of 63 percent.

4. An idealized basin designed for a rate equal to 20 percent of the 10-year design storm can be designed to achieve an average suspended solids removal of 81 percent.

SECTION 2

RECOMMENDATIONS

1. The analyses of combined sewer overflow systems should take into account the frequency of occurrence of overflow rate, so that proper weight can be assigned to flows in the analysis of hydraulic loading of treatment systems or conveyance systems. Systems must be designed to operate most efficiently under prevailing conditions and still have the capacity to handle peak flows.

2. Considering the extremely high coliform counts in the discharge area of the Milk River drain into Lake Saint Clair and considering the use of the water for limited, body contact recreational purposes, it is recommended that the combined sewer overflow from the Milk River Pumping Station be treated with sodium hypochlorite for significant reduction of bacterial levels.

3. One of the 305 cfs (137,000 gpm) storm pumps at Milk River should be replaced with a flexible pumping system having a variable capacity of from approximately 100 cfs through 305 cfs, to decrease the average rate of flow through the retention basin during most storm events.

4. At least two weirs should be placed across the width of the existing Milk River retention basin to increase the effect of retention in the basin by reducing the effect of horizontal short-circuiting and recirculation.

5. Major components of chemical feed equipment utilized on this Contract should be incorporated, along with a new flocculator, the recommended weirs and variable speed pumping to increase the suspended solids removal to approximately 39 percent at the Milk River Pumping Station.

6. Analytical data should be developed on the separated storm sewage from Harper Woods and compared with the combined sewage of Grosse Pointe Woods. Both areas have similar population, topography, area, and residential character, thereby providing the means by which the effect of sanitary sewage on storm runoff could be measured.

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SECTION 3

INTRODUCTION

Most sanitary sewerage systems in the United States collect not only domestic sewage and industrial wastes, but also receive large quantities of surface runoff during periods of significant rainfall. In 1967, the Federal Water Quality Administration of the U. S. Department of Interior estimated that of the approximately 200,000,000 persons in this country, about 125,000,000 (62.5%) were served by combined or separate sanitary sewers. They also estimated that approximately 36,000,000 of these persons were connected to sewerage systems which collect not only sanitary wastes but also all or part of the storm runoff from the areas which they serve[1].

There are two main reasons for the widespread existence of these combined sewers. Firstly, enclosed storm sewers preceded public water supplies and when central plumbing came into general use, the logical points of discharge for household wastes were the already existing storm sewers. Secondly, even after the water carriage principle of sewage disposal was adopted, the short-term cost of constructing one sewer, usually just slightly larger than that required for storm flow, was considerably less than the cost of constructing separate sewers for sanitary and storm sewage.

Since it is not economically feasible to construct adequate treatment facilities to care for the entire flow of combined systems for all rainfall conditions, it has been common practice to restrict the flow in interceptor and other main sewers to some predetermined value. During peak storms, the rate of runoff can approach 100 times the dry weather sanitary flow but most sewerage systems are designed to contain a maximum of about eight times, or more frequently about four times the dry weather flow[2]. Relief structures are provided at suitable points along these sewers to permit the excess combined sewage to over flow to available water courses.

These overflows of sanitary sewage mixed with stormwater contribute significantly to the pollution and deterioration of our water resources. Because of the extreme variability of rainfall and runoff patterns, overflow structure designs, maintenance practices, sewer construction standards, the existence of other major sources of pollution, and numerous other important factors, it is extremely difficult to quantitate the overall extent and effect of this source of water pollution. To further complicate matters, the compositions of most combined sewer system discharges vary considerably with time during a single storm and from storm to storm.

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The first flush of a significant storm carries large quantities of loose suspended and soluble material from pavements and landscaped areas[3,4,5]. Street deicing compounds are washed from pavements during winter thaws. These materials are added to the sanitary flow along with the large volumes of stormwater which may resuspend additional accumulations of grit, sludge and other material previously settled in catch basins and sewers[6]. Large volumes of this mixture then overflow into convenient water courses causing serious bacterial pollution, aesthetic deterioration, and often undesirable biological changes in the receiving waters[7].

At least three general solutions to this pollution problem are available.

1. The sanitary and storm sewers can be separated.
2. The combined sewage overflow can be collected in some type of reservoir and drained back to the sewage plant for treatment during dry weather.
3. The combined sewage overflow can receive some appropriate type of treatment allowing water of acceptable quality to be discharged to the receiving waters.

In 1967, the Federal Water Quality Administration estimated that the total cost to separate the combined sewers in this country on both public and private property would be approximately $48 billion[2]. These estimates make it quite clear that separation of sanitary and storm sewers, while still the most desirable solution where feasible, is not a general solution to the combined sewer overflow problem.

Numerous public agencies and private enterprises are currently studying various methods and processes for solution of the problem, utilizing the principles of storage with subsequent treatment before discharge. Because of the variability in volume, pollution load, and relatively infrequent occurrence of combined sewer overflows, it appears reasonable to expect the most economical systems to have wide flexibility, low capital cost, and an operating cost that can vary with the load. Because of the cost and limited availability of land in the areas of greatest need, economics would appear to favor smaller plants. Some of these advantages are available in systems utilizing chemical treatment.

During 1968 and 1969, The Dow Chemical Company, through Project No. 11030 FDB with the Federal Water Quality Administration of the Department of Interior, was involved in a comprehensive research and development program aimed at evaluating the applicability of various available chemicals for treatment of a major combined sewer overflow. The work was carried out at the Milk River Pumping Station (MRPS) in Grosse Pointe Woods, Michigan, in cooperation with the intercounty Milk River Drainage Board.

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SECTION 4

DESCRIPTION, HISTORY AND DEVELOPMENT OF THE MILK RIVER DRAINAGE BASIN

Before reviewing the results of the experimental work conducted for the Milk River Project, it is extremely important to gain an understanding of the Milk River combined sewer overflow problem. This understanding provides the framework for a rational analysis of the problem and outlines some of the major considerations which must be taken into account in the ultimate design of combined sewer overflow treatment facilities. Following is a description of the character of the area within the limits of the drainage basin and of the system by which the storm and sanitary sewage from the area is conveyed and handled.

DESCRIPTION OF THE MILK RIVER DRAINAGE AREA

The Milk River Drainage Basin, comprised almost entirely of the cities of Harper Woods and Grosse Pointe Woods, is an essentially completely developed residential area of approximately 3990 acres located in suburban northeast Detroit, Michigan. Both cities are zoned for limited local retail business which is concentrated along four main arterial highways, Vernier Road (M-29), Mack Avenue, Kelly Road and the Northerly Extension of the Edsel Ford Expressway (I-94). The only other notable physical features of the basin are a private golf course (Lochmoor Country Club), a large retail business area (Eastland Shopping Center) and several relatively smaller school sites and municipal parks.

As can be seen from Figure 1, the topography of the basin is quite flat and generally slopes toward the Milk River channel which runs northeasterly between Mack Avenue and Lake Shore Drive into the community of Saint Clair Shores. The topography of the basin generally falls easterly toward the Milk River channel from a maximum elevation of approximately 600 feet above sea level (USGS datum) along Kelly Road in Harper Woods and falls northwesterly from an elevation of 585 to 590 feet above sea level toward the Milk River channel from a ridge line running parallel to and west of Lake Shore Drive in Grosse Pointe Shores. The Milk River channel is the lowest point in the drainage area with an elevation of approximately 575 along the banks of the channel at the point where the Milk River crosses into Saint Clair Shores between Mack Avenue and Lake Shore Drive along the northerly limit of the drainage district.

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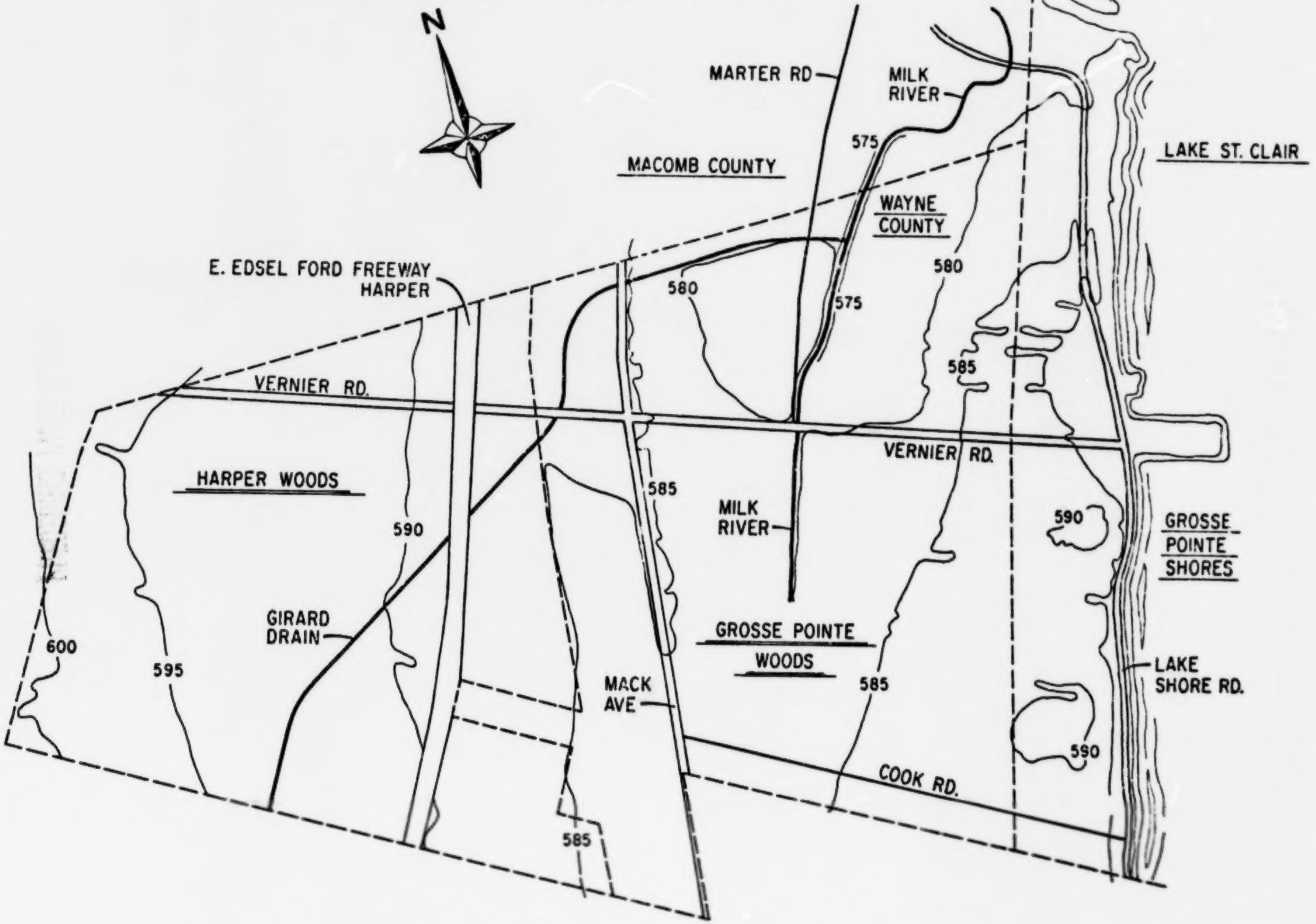
Figure 1

Topographical Map of Milk River Drainage Basin


N
MARTER RD
MILK RIVER
575
LAKE ST. CLAIR
MACOMB COUNTY
WAYNE COUNTY
580
E. EDSEL FORD FREEWAY HARPER
580
575
585
VERNIER RD.
VERNIER RD.
HARPER WOODS
585
MILK RIVER
590
590
GROSSE POINTE SHORES
GIRARD DRAIN
GROSSE POINTE WOODS
600
595
MACK AVE
585
LAKE SHORE RD.
590
COOK RD.
585

POPULATION OF THE MILK RIVER DRAINAGE BASIN

Following is a summary of data on population growth for the cities of Grosse Pointe Woods and Harper Woods.

TABLE I

POPULATION OF HARPER WOODS AND
GROSSE POINTE WOODS (1920-1926)

(U. S. Bureau of Census)

Year	Harper Woods	Grosse Pointe Woods
1920	73 (Gratiot)	961 (Lochmoor)
1940	858	2,805
1950	9,148	10,381
1960	19,995	18,580

As can be seen from the data, population growth in both communities was very rapid between the years 1940-1960. Development in both communities for the years for which the Milk River Pumping Station has been in operation can be seen in Table II. These data were taken from the publication, Population and Occupied Dwelling Units in The Detroit Region, compiled by the Southeast Michigan Council of Governments.

These data, coupled with the U. S. Bureau of the Census information show that maximum development of the communities occurred between 1940-1960. As noted previously, both Grosse Pointe Woods and Harper Woods are almost completely developed based on present zoning standards. This is reflected in the decreasing number of new dwelling units being occupied each year. The present population density of the area is approximately 11.2 persons per acre.

HISTORY AND DEVELOPMENT OF WASTEWATER HANDLING FACILITIES IN THE MILK RIVER DRAINAGE BASIN

The original topography of the basin had several natural watercourses which served as the drainage system for surface water runoff (Figure 1). Drainage for the area that is now Harper Woods followed a small shallow ditch known as the Girard Drain. This stream originated in the southwest area of Harper Woods and flowed northeasterly across the city into the northern section of the present city of Grosse Pointe Woods. A second watercourse, the Black Marsh Drain, flowed northerly from the southerly part of what is now Grosse Pointe Woods to a point south of Lochmoor Road. The watercourse at this point became known as the Milk River Drain.

TABLE II

POPULATION OF HARPER WOODS AND GROSSE POINTE WOODS (1961-1968)

Year - as of July 1	HARPER WOODS		GROSSE POINTE WOODS		TOTAL	
	Population	Dwelling Units	Population	Dwelling Units	Population	Dwelling Units
1961	20,100	5,410	18,800	5,300	38,900	10,710
1962	20,600	5,550	19,300	5,450	39,900	11,000
1963	20,500	5,750	19,900	5,600	40,400	11,350
1964	20,800	5,950	21,100	5,880	41,900	11,830
1965	22,000	6,230	21,200	6,080	43,200	12,310
1966	22,200	6,200	21,600	6,120	43,800	12,320
1967	22,600	6,300	22,000	6,240	44,600	12,540
1968	22,650	6,320	22,200	6,290	44,850	12,610

The Milk River continues to flow northward and joins with the Girard Drain just east of Marter Road at the north edge of Grosse Pointe Woods. From this point, it meanders in a generally northeasterly direction for approximately 6000 feet through Macomb County in what is now the city of Saint Clair Shores and finally discharges into Lake Saint Clair just south of the extension of Nine Mile Road. None of these streams carried any appreciable quantity of dry weather flow but were filled to some extent by water which backed up from Lake Saint Clair.

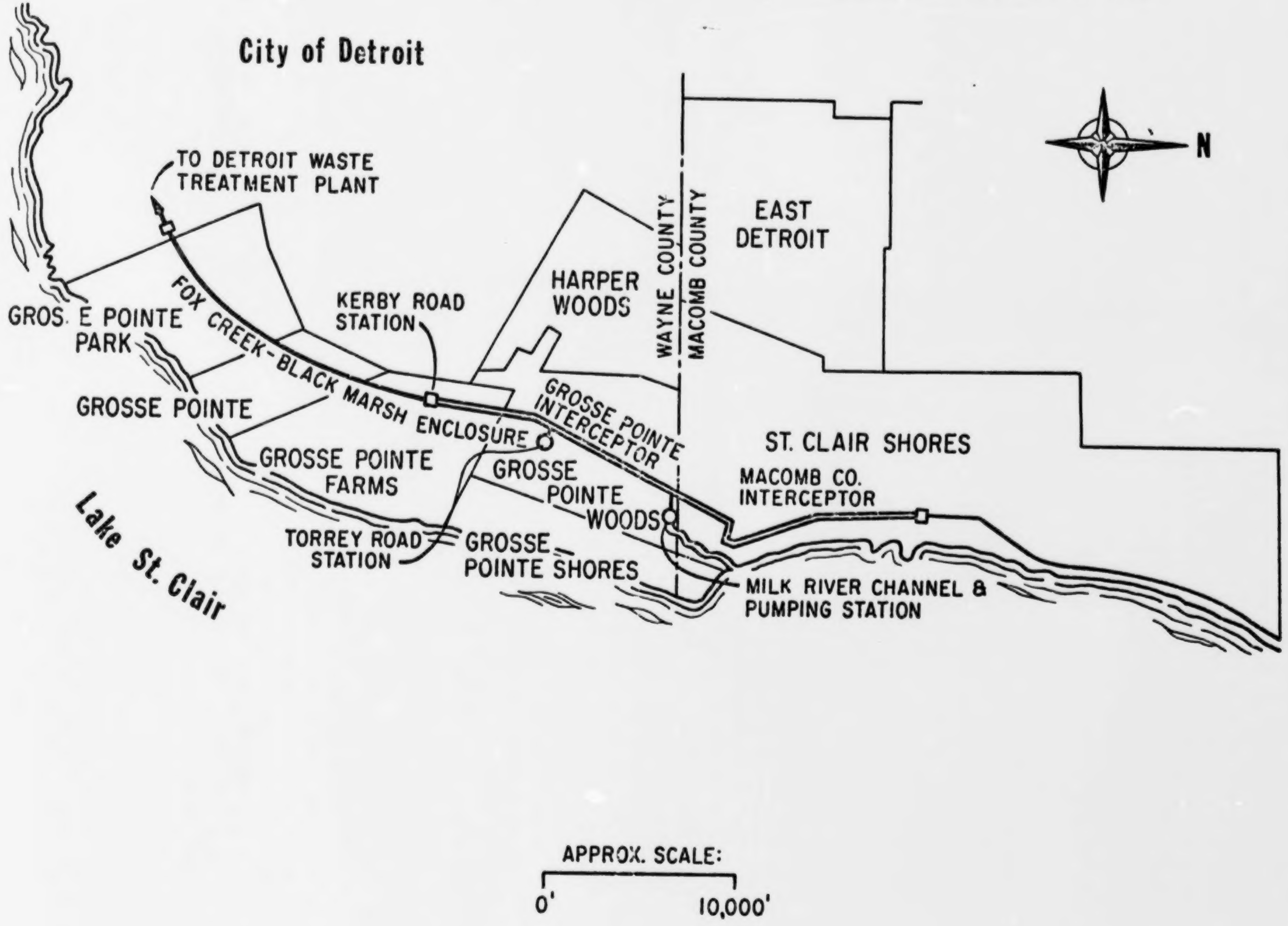
Prior to construction of the Milk River Pumping Station, the Black Marsh and Milk River drains served as the point of discharge for the combined sewage from the community of Grosse Pointe Woods. This sewage was collected in a combined sewer system, part of which was designed and constructed prior to 1929. These early improvements committed the city of Grosse Pointe Woods to a course of using combined sewerage facilities. About 98 percent of the sewers are presently combined in the community with almost saturated development by present zoning standards.

The city of Harper Woods, in comparison, was initially developed in an area adjacent to the city of Detroit. Sanitary sewers were constructed to discharge into the Detroit collection system. As the area in the Milk River drainage district developed, Harper Woods remained committed to separate sewers. At the present time, with almost saturated development, approximately 90 percent of the sanitary wastes from the community is collected in separate sanitary sewers.

In 1942, a 72-inch sanitary interceptor was constructed through the central part of Grosse Pointe Woods (see Figure 2). This interceptor, paralleling the basic courses of the Black Marsh-Milk River Drains, extended north to Eight Mile Road (Wayne-Macomb County line) from what is known as the Fox Creek Enclosure in Grosse Pointe Farms. The Kerby Road pumping station, located at the Fox Creek enclosure and Grosse Pointe Woods Interceptor Junction was modified to lift the incoming flow from the Grosse Pointe Woods interceptor to the Fox Creek enclosure for eventual discharge into the Detroit sewerage system. Construction of this interceptor permitted most of the combined sewers in Grosse Pointe Woods to be connected to the Detroit sewerage system, thus eliminating most of the discharge of dry weather sanitary flow into the open drains.

Figure 2

SANITARY WASTE DISPOSAL FOR AREAS NORTHEAST OF DETROIT


City of Detroit
N
TO DETROIT WASTE TREATMENT PLANT
EAST DETROIT
WAYNE COUNTY
MACOMB COUNTY
HARPER WOODS
KERBY ROAD STATION
GROS. E POINTE PARK
FOX CREEK-BLACK MARSH ENCLOSURE
GROSSE POINTE
GROSSE POINTE INTERCEPTOR
ST. CLAIR SHORES
GROSSE POINTE FARMS
GROSSE POINTE WOODS
MACOMB CO. INTERCEPTOR
Lake St. Clair
TORREY ROAD STATION
GROSSE POINTE SHORES
MILK RIVER CHANNEL & PUMPING STATION
APPROX. SCALE:
0'
10,000'

During the three years following construction of the 72-inch interceptor, both the cities of Grosse Pointe Woods and Harper Woods initiated considerable construction of new sewers to improve their systems. In Grosse Pointe Woods, a new pumping station was constructed at Torrey Road, and improvements were made to an existing station at Marter Road and Hollywood Avenue. Both stations were designed to discharge dry weather sanitary flow by gravity into the 72-inch interceptor.

The Torrey Road Pumping Station collected almost all of the sanitary sewage from Harper Woods and some of the combined sewage from Grosse Pointe Woods. During periods of heavy rainfall when the interceptor was filled to capacity, the sewage would back up into the Station until automatic controls started storm pumps which discharged the combined sewage into the open Milk River Drain. The Torrey Road Station had three storm pumps with a combined capacity of 275 cfs (124,000 gpm) and the Marter Road Station had a pumping capacity of approximately 200 cfs (89,800 gpm).

The rapid increase in population of both cities required major improvements to be made to their trunk sewers beginning in 1954. It was estimated at this time that about three-fourths of the ultimate population of these communities had been reached. The Milk River and Black Marsh Drains were becoming increasingly septic from combined wastewater overflow discharges. In 1956, two additional pumps and a weir gate were installed in the Torrey Road Pumping Station by order of the Michigan State Board of Health. The pumps were installed to permit discharge at the first flush of combined sanitary sewage and storm runoff into the interceptor with the weir gate closed.

It had become apparent by this time that major construction was required to alleviate wastewater handling problems in the area. The pollutional problems associated with the combined wastewater overflowing into the open drains leading to Lake Saint Clair were of particular concerns. In 1958, construction was started on collector sewers which generally followed the course of the Black Marsh, Milk River and Girard Drains. These sewers were designed to collect almost all of the sanitary, storm and combined flows from the communities of Harper Woods and Grosse Pointe Woods. This flow was then discharged to a new pumping station built at the intersection of the Milk River and Girard Drains. The facility, called the Milk River Pumping Station (MRPS), was designed to pump the dry weather flow from the two communities into the 72-inch interceptor by means of three sanitary sewage pumps each having a capacity of about 11.1 cfs (5,000 gpm).

During storm periods, the combined sanitary and storm sewage from both Grosse Pointe Woods and Harper Woods is lifted by means of one or a combination of from two to seven storm pumps into an open concrete settling and skimming basin adjoining the pumping station, the effluent from which overflows into the Milk River Drain and eventually into Lake Saint Clair. A flow diagram of the system is shown in Figure 3 and a view of the settling basin is shown in Figure 4.

The Torrey Road Pumping Station remained as part of the new system. Until 1968, it continued to discharge the dry weather from from Harper Woods and part of Grosse Pointe Woods directly into the 72-inch interceptor. The storm overflows, which originally discharged into the open drain, were now transported directly to the MRPS. In 1968, this operation was revised and the Torrey Road Pumping Station now discharges both dry weather and storm flows directly to the MRPS. The present major storm, sanitary, and combined sewers of Grosse Pointe Woods and Harper Woods are shown in Figures 5 and 6.

The operation of the system can be summarized. Prior to 1968, the MRPS handled principally dry weather flow from the combined system of Grosse Pointe Woods and lifted it into the 72-inch interceptor. After 1968 the Harper Woods dry weather sanitary flow was pumped from the Torrey Road Pumping Station to the MRPS. The MRPS serves as a sanitary lift station to the interceptor for all dry weather flow from the communities of Grosse Pointe Woods and Harper Woods. During storm periods, the Station serves as a combined sewer overflow pumping station for all stormwater from Harper Woods, combined wastewater from Grosse Pointe Woods and sanitary wastewater from Harper Woods which cannot be pumped into the interceptor during storm periods.

BASIS OF DESIGN OF THE MILK RIVER TRUNK SEWERS

In general, combined and storm sewers in the Milk River Drainage Basin area have been sized for runoff calculated according to the "rational" method.

This method is usually expressed as:

$$Q = CIA$$

Q = Rate of runoff, cubic feet per second*
C = Runoff coefficient, % imperviousness
I = Intensity of the rainfall, inches per hour
A = Drainage area, acres

*1 cfs = 448.8 gpm

Figure 3

FLOW DIAGRAM – MILK RIVER OVERFLOW SYSTEM
(Not To Scale)

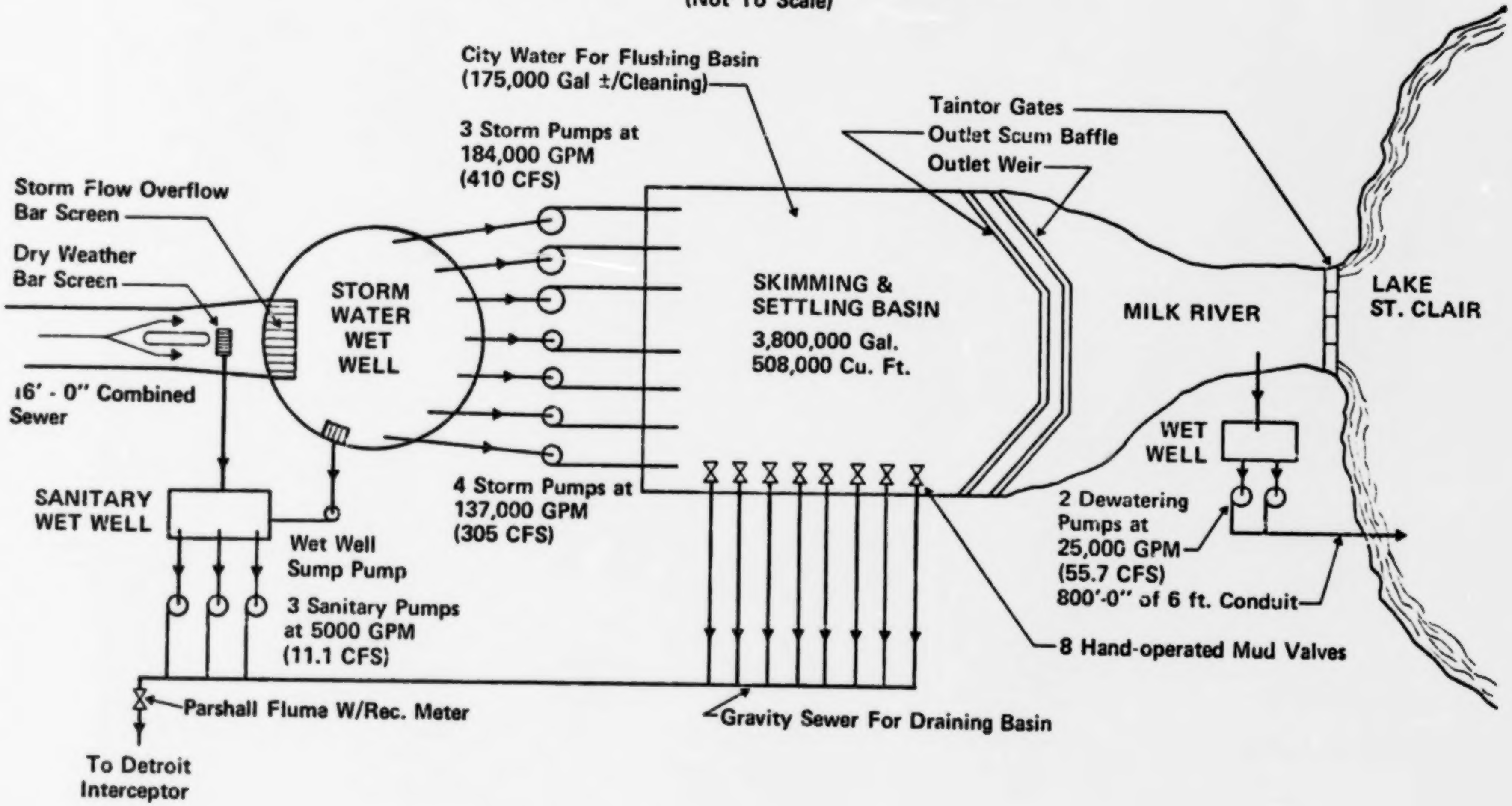

City Water For Flushing Basin
(175,000 Gal ±/Cleaning)
3 Storm Pumps at
184,000 GPM
(410 CFS)
Taintor Gates
Outlet Scum Baffle
Outlet Weir
Storm Flow Overflow
Bar Screen
Dry Weather
Bar Screen
STORM
WATER
WET
WELL
SKIMMING &
SETTLING BASIN
3,800,000 Gal.
508,000 Cu. Ft.
MILK RIVER
LAKE
ST. CLAIR
16' - 0" Combined
Sewer
WET
WELL
SANITARY
WET WELL
4 Storm Pumps at
137,000 GPM
(305 CFS)
Wet Well
Sump Pump
2 Dewatering
Pumps at
25,000 GPM
(55.7 CFS)
800'-0" of 6 ft. Conduit
3 Sanitary Pumps
at 5000 GPM
(11.1 CFS)
8 Hand-operated Mud Valves
Parshall Fluma W/Rec. Meter
Gravity Sewer For Draining Basin
To Detroit
Interceptor

Figure 4
Overflow System Basin at Milk River Pumping Station

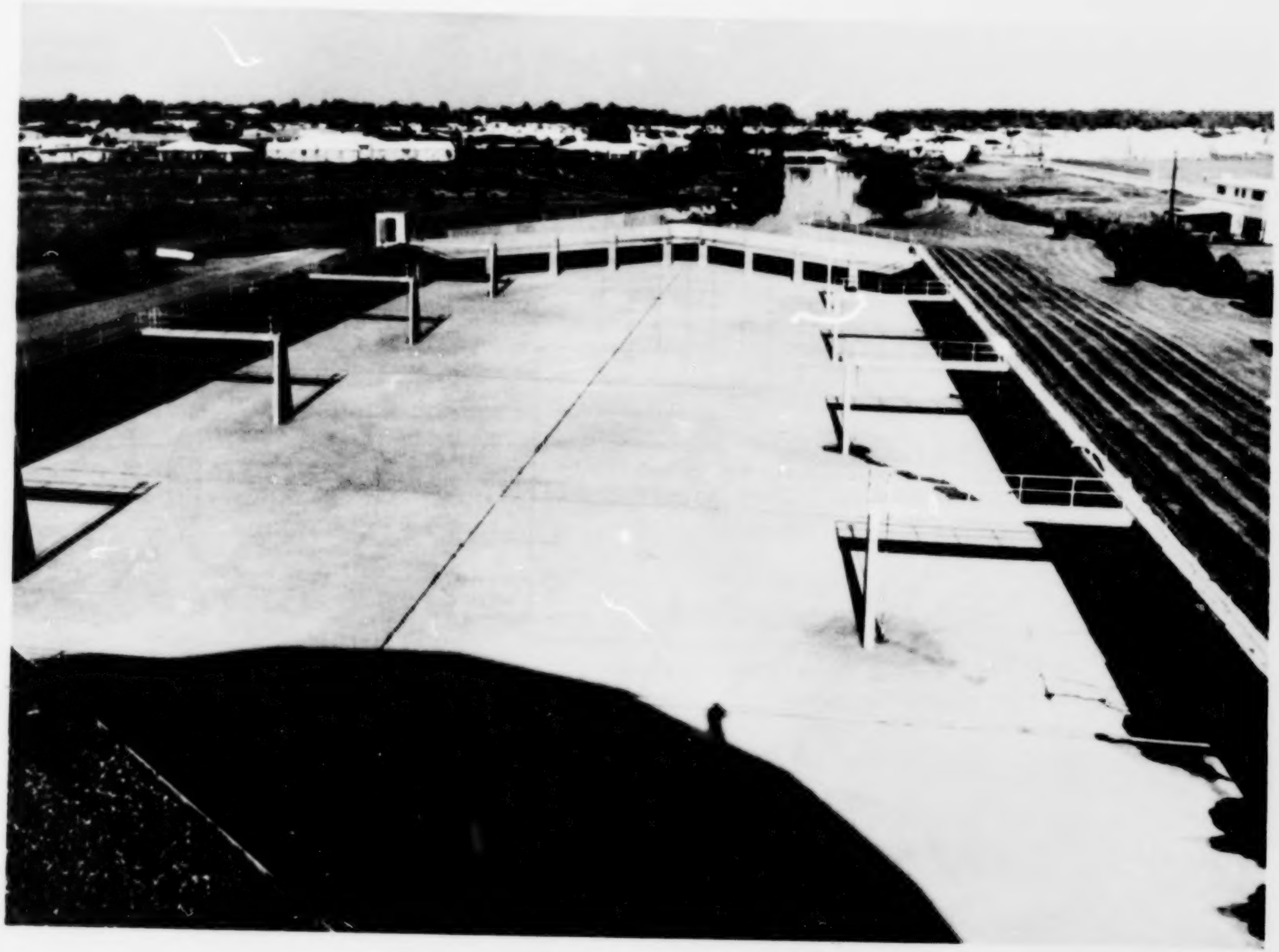


Figure 5

HARPER WOODS SEPARATE SANITARY &
GROSSE POINTE WOODS COMBINED SEWERAGE SYSTEM

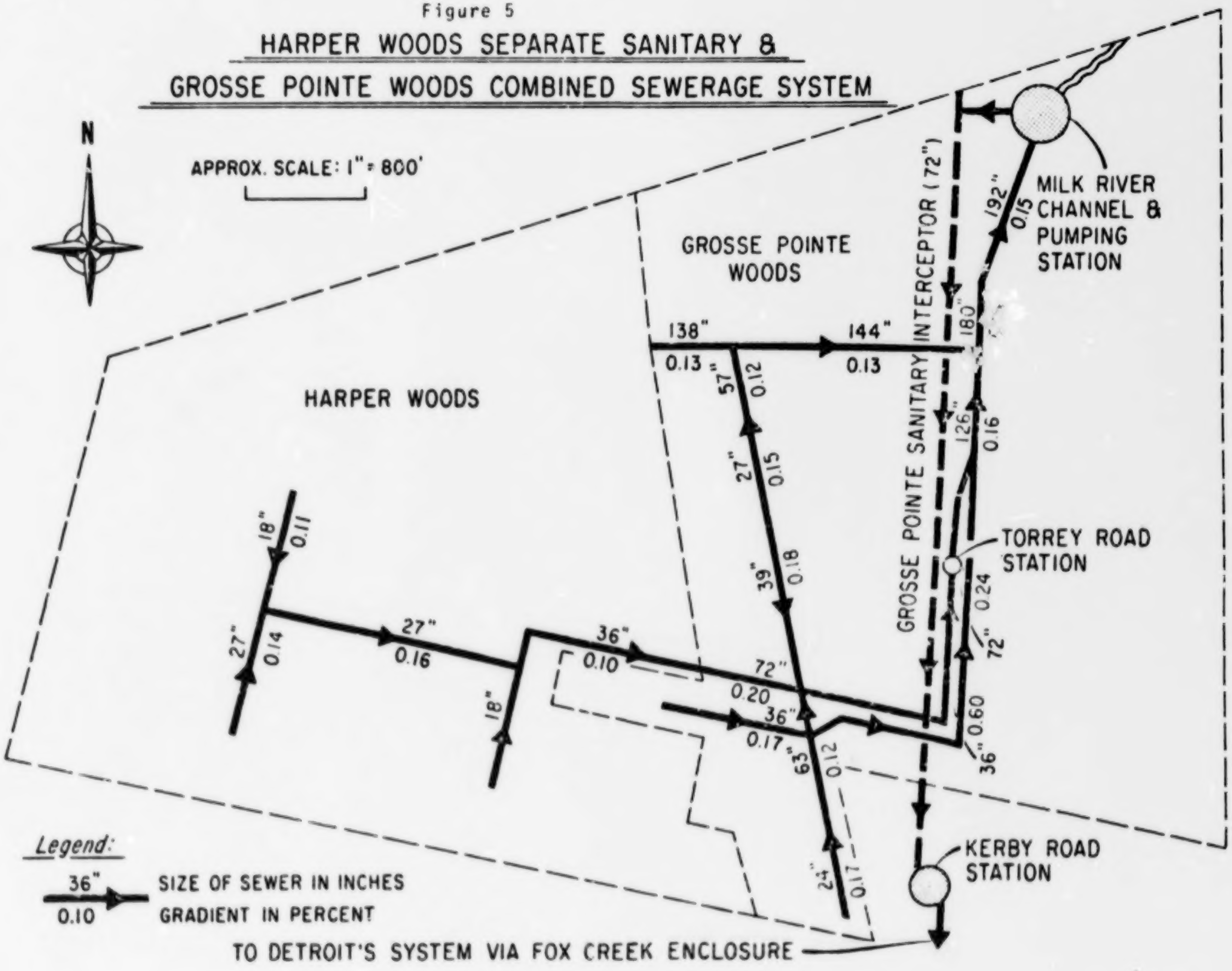
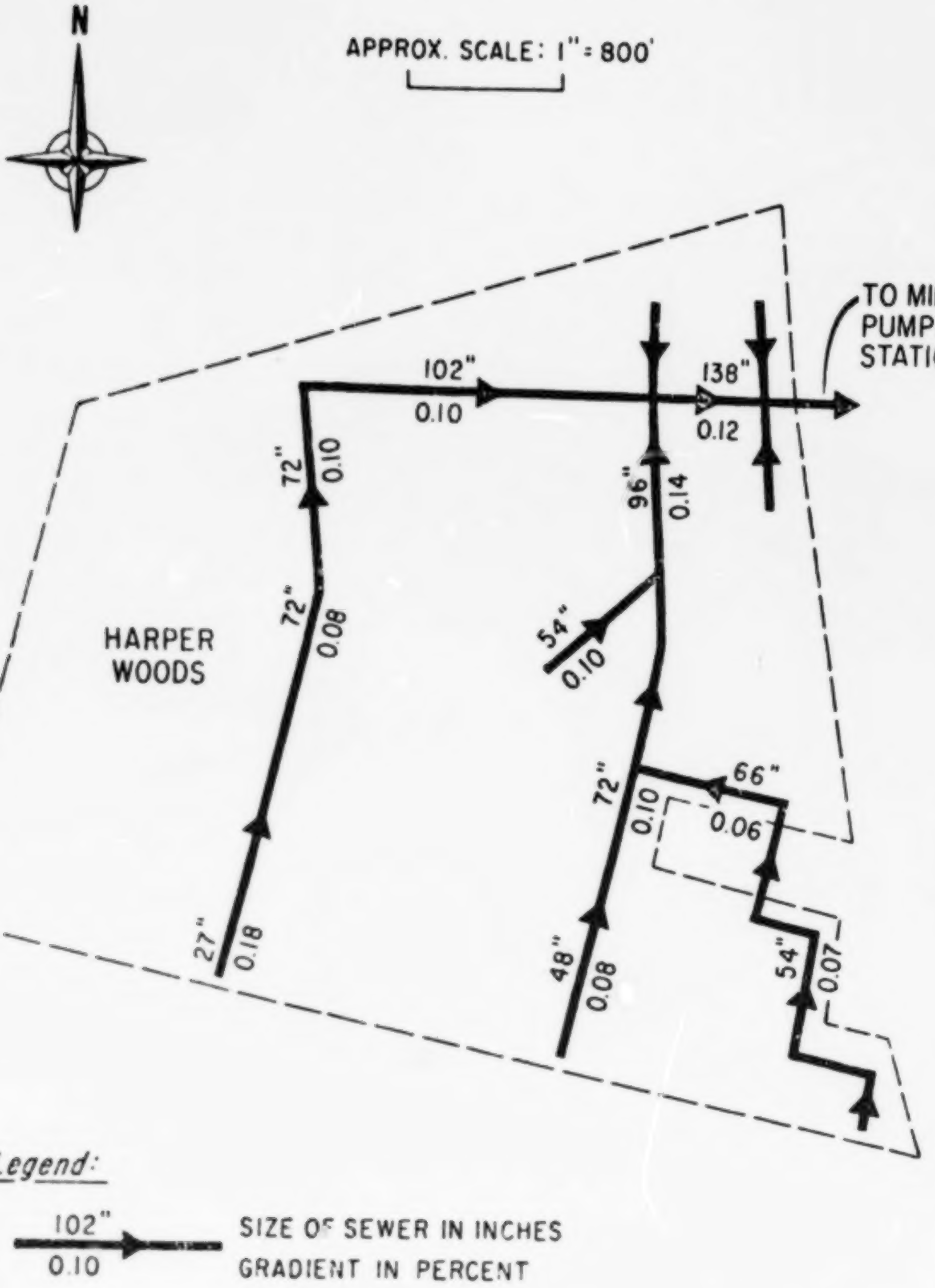

N
APPROX. SCALE: 1" = 800'
HARPER WOODS
GROSSE POINTE WOODS
GROSSE POINTE SANITARY INTERCEPTOR (72")
MILK RIVER CHANNEL & PUMPING STATION
TORREY ROAD STATION
KERBY ROAD STATION
192"
0.15
180"
138"
0.13
144"
0.13
57"
0.12
27"
0.15
39"
0.18
126"
0.16
72"
0.24
36"
0.60
18"
0.11
27"
0.14
27"
0.16
18"
36"
0.10
72"
0.20
36"
0.17
63"
0.12
24"
0.17
Legend:
36"
0.10
SIZE OF SEWER IN INCHES
GRADIENT IN PERCENT
TO DETROIT'S SYSTEM VIA FOX CREEK ENCLOSURE

Figure 6

STORMWATER TRUNK SEWERS OF HARPER WOODS


N
APPROX. SCALE: 1" = 800'
102"
0.10
138"
0.12
TO MILK RIVER PUMPING STATION
72"
0.10
96"
0.14
HARPER WOODS
72"
0.08
54"
0.10
72"
0.10
66"
0.06
27"
0.18
48"
0.08
54"
0.07
Legend:
102"
0.10
SIZE OF SEWER IN INCHES
GRADIENT IN PERCENT

Harper Woods storm sewers are designed to carry the runoff from a storm of an intensity which would most probably occur once in five years. The Grosse Pointe Woods combined sewer system is designed to carry the runoff from a storm of an intensity which most probably would occur once in ten years.

In the design of the MRPS and trunk sewers, the calculated runoff from Harper Woods was adjusted for the ten-year frequency rainfall at the point of entry into the Grosse Pointe Woods system. In estimating the runoff, intensity curves similar to those developed by the city of Detroit for ten-year frequency storms were used. Specifically, rainfall intensity was calculated to be:

$$I = \frac{164}{t + 26.5}$$

where I = Intensity, inches per hour
t = Duration of the storm, minutes

A runoff coefficient of 0.20 was used for all park, school and church areas. All other areas were considered to have a runoff coefficient of 0.30.

The design engineers calculated the time of concentration at various points in the sewer system by estimating the average velocities in existing sewers to be approximately 3.0 fps and by computing the velocities in the trunk sewers using the calculated runoff and gradients for the proposed trunk sewers. An inlet time of 20 minutes was assumed. Based on these assumptions, the time of concentration, including inlet time, was calculated for the following locations:

TABLE III

MILK RIVER COMBINED SEWER TIME OF CONCENTRATION[1]

Location	Time of Concentration (Minutes)
Torrey Road Pumping Station	61.0
Vernier and Mack	84.0
Vernier Road and Milk River Drain	89.0
Milk River Pumping Station	92.0

The calculated rate of total runoff at the MRPS based on a ten-year frequency storm, was calculated by the design engineers to be 2450 cfs. From these calculations, the influent trunk sewer diameter required to handle the design flow of 2450 cfs (1,100,000 gpm) was calculated to be 16 feet at an average gradient of 0.15 percent.

ADMINISTRATION OF WASTEWATER HANDLING FACILITIES IN THE MILK RIVER DRAINAGE AREA

In June of 1955, under the provisions of the State of Michigan Drain Acts, the cities of Harper Woods and Grosse Pointe Woods petitioned the Wayne County Drain Commission to finance, construct and operate major sewerage facilities connected with:

(a) The Girard Drain
(b) The Black Marsh Drain
(c) The Milk River Drain

As seen from Figure 1, the Girard and Black Marsh Drains are located entirely within Wayne County. The Milk River Drain, on the other hand, flows through the city of Saint Clair Shores in Macomb County. Under the provisions of the law, intra-county drainage boards were established for the Girard and Black Marsh Drains and an inter-county drainage board established for the Milk River Drain.

The Inter-County Drainage Board is made up of the Drain Commissioner of Wayne County, the Drain Commissioner of Macomb County and one representative from the State of Michigan. This group, called the Milk River Drainage Board, administers the Milk River Drain including operation of the MRPS and other facilities, while the Wayne County Drain Commission actually operates the MRPS. The Torrey Road Pumping Station is still operated by the city of Grosse Pointe Woods. Most other facilities, except for the trunk sewers constructed under petition to the Wayne County Drain Commission are operated by the cities of Grosse Pointe Woods and Harper Woods.

SECTION 5

BACKGROUND DATA AND ANALYSIS

Understanding of the problem and identification of significant variables at the Milk River Pumping Station (MRPS) are necessary before rationalizing the potential of treatment by any chemical systems. The significant variables include, but are not restricted to, rainfall-runoff patterns, runoff rates and volumes, and runoff quality.

LONG-TERM RAINFALL-RUNOFF PATTERNS

The long-term relationships between rainfall, runoff, and operation of the pumping station must be understood before studying specific rainfall, runoff and quality data. These relationships can best be seen in Table IV and in Figure 7. The relationships between rainfall, runoff, stormwater pumping, and sanitary sewage pumping at the MRPS is tabulated for eight years.

Several comments are in order:

1. Only calendar years of records are shown.
2. Total rainfall for the basin was calculated using data from the Detroit City Airport rain gauge station.
3. Dry weather flow was calculated using the lowest seven-day average for each year.

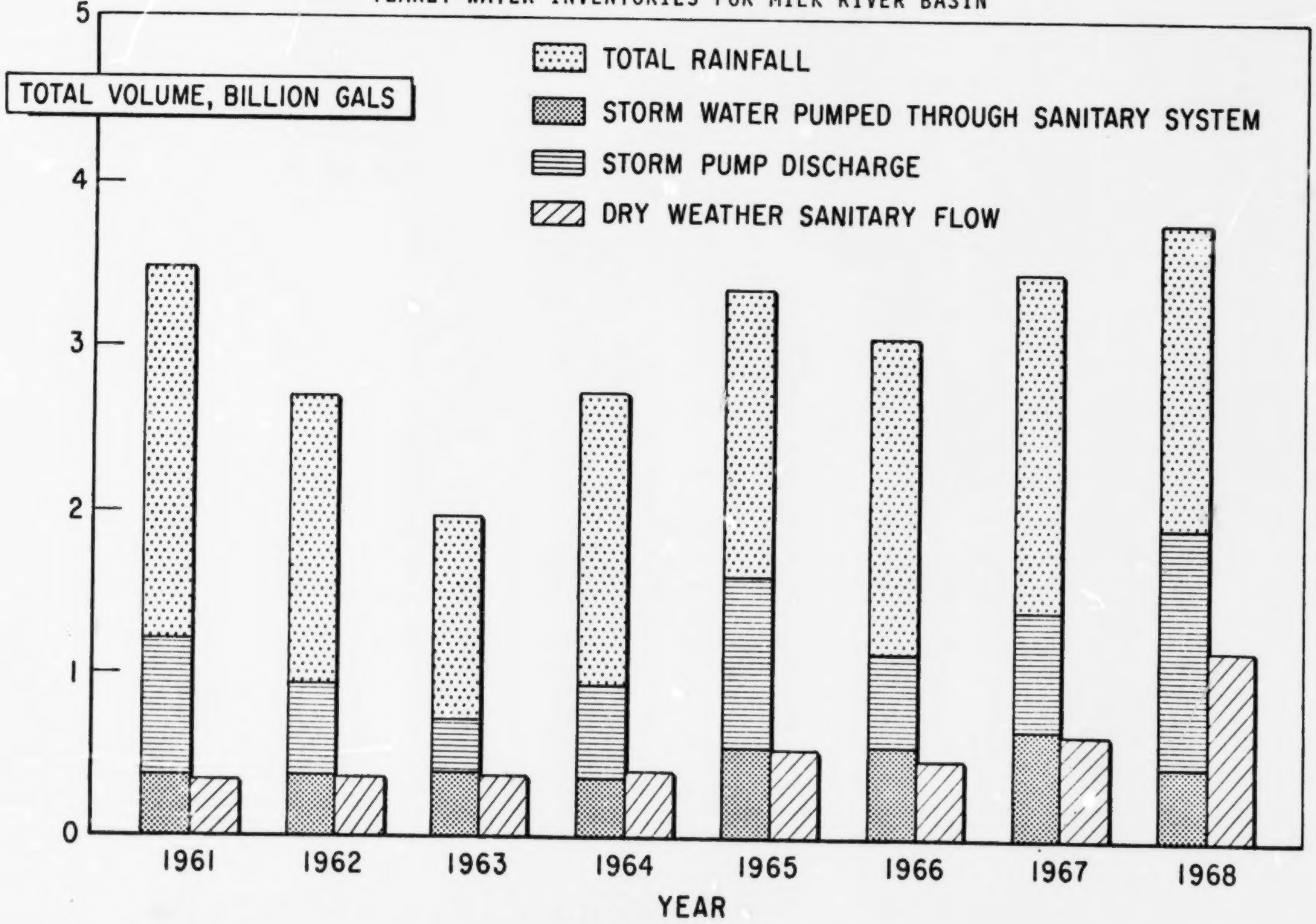
The total yearly volume of stormwater discharged to Lake Saint Clair significantly is almost equal to that pumped from the MRPS through the Detroit Interceptor sewer. The total annual rainfall varied by almost 50 percent within the eight years shown; the average yearly runoff varied from 34 percent to 51 percent. These differences reflect the extreme variability in storm patterns and indicate the necessity for evaluating and analyzing runoff data from selected storms rather than from averaged hydrological data. Runoff coefficients for project storms of record varied from 15 percent to 60 percent. Rainfall and pumping hydrographs have been constructed for each project record storm to establish individual storm runoff hydrograph patterns.

TABLE IV

SUMMARY OF TOTAL YEARLY FLOWS IN MILK RIVER BASIN (1961-1968)

Year	Cumulative Rainfall (Inches)	Cumulative Rainfall (10^6 Gal)	Total Storm Pump Discharge (10^6 Gal)	Total Sanitary Flow Pumped (10^6 Gal)	Estimated Total Dry Weather Flow (10^6 Gal)	Stormwater Pumped Thru San. System (10^6 Gal)	Total Storm Runoff (10^6 Gal)	% Runoff
1961	33.80	3,490	813	711	325	386	1,199	0.34
1962	26.51	2,740	575	671	327	344	919	0.34
1963	19.15	1,980	331	739	341	398	729	0.37
1964	26.39	2,720	575	735	389	346	921	0.34
1965	32.47	3,350	1,047	1,072	533	539	1,586	0.47
1966	29.31	3,060	586	1,041	478	563	1,149	0.37
1967	33.83	3,479	738	1,287	630	657	1,395	0.40
1968	36.63	3,780	1,471	1,598	1,160	438	1,909	0.51
Average	29.76	3,075	767	982	523	459	1,226	0.39

Figure 7
YEARLY WATER INVENTORIES FOR MILK RIVER BASIN



MEASUREMENT OF RAINFALL AND RUNOFF

The rainfall hydrographs were constructed from data obtained from three weight-type rainfall gauges located within the Milk River Drainage area. Each of these three rainfall stations is assumed to have a weighted effect upon the average rainfall for the entire drainage area according to a method suggested by Thiessen[1].

The location of each of the three gauging stations in the drainage area is shown in Figure 8. They are designated W-31 (Torrey Road Pumping Station), W-32 (Detroit Edison Eastland Substation), and W-33 (Milk River Pumping Station). These stations were selected for several reasons: good areal coverage, optimum site conditions, ready accessibility for maintenance and checking, and location within fenced areas under the direct supervision of a responsible agency. The locations were inspected by the ESSA, found acceptable as gauging sites, and incorporated as part of the Detroit Regional Rain Gauge Network. The station charts are processed and evaluated by the Office of the State Climatologist in Lansing as part of the Regional Network Program and the data published in the monthly rainfall reports of the SEMCOG Network.

The calculated station constants (Thiessen method) for each of the three program gauging locations are presented in Table V. These constants are applied to the point rainfall data in the calculation of the rainfall hydrographs.

TABLE V

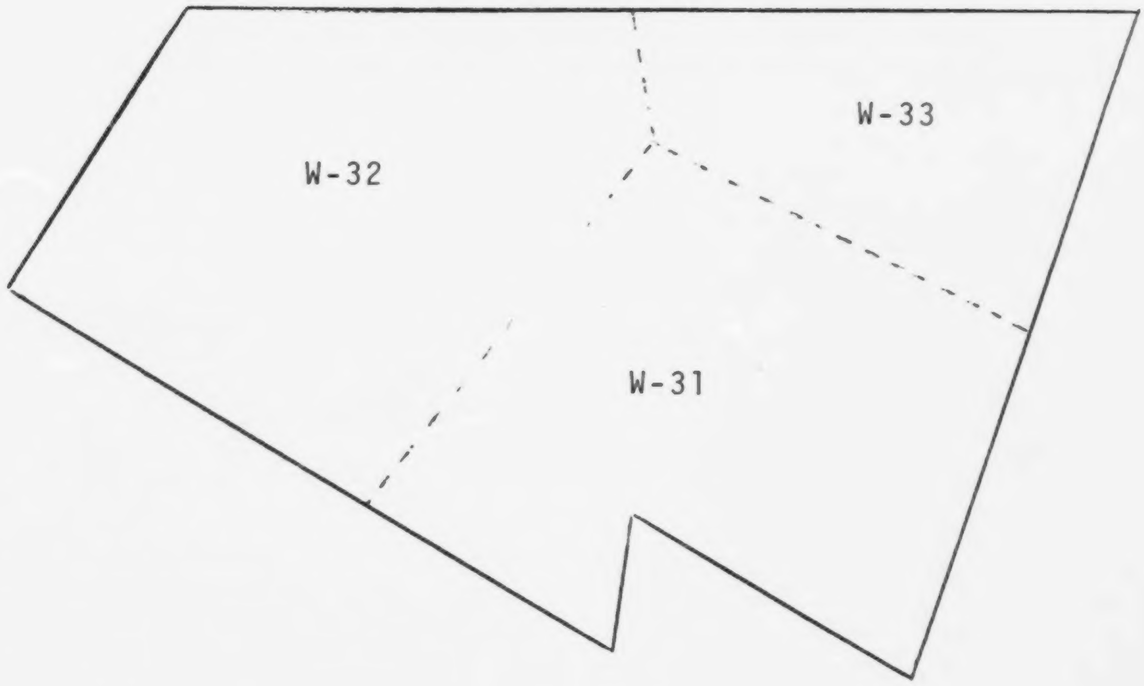
CALCULATED STATION CONSTANTS

Station	Area (Thiessens Method)	Station Constant
W-31	1572 acres	0.39
W-32	1616	0.41
W-33	802	0.20
	3990	1.00

Storm runoff hydrographs were constructed using a method that provided an acceptable degree of accuracy using existing Station metering equipment. The method selected permitted us to easily analyze the pumping data, including all storm overflows from the time that the Station was placed in operation.

Figure 8

OUTLINE OF THE MILK RIVER DRAINAGE AREA



The seven axial flow storm pumps at Milk River have propeller type impellers. Three have a nominal capacity of 410 cfs (184,000 gpm) and four have a nominal capacity of 305 cfs (137,000 gpm). The exact capacity is a function of the total head against which the impeller is pumping at the time. At Milk River, the change in total head is determined by the change in suction head, because the discharge elevation is fixed. Since the elevation of the water in the storm well is continuously recorded, the change in total head on the pump can be determined. The elapsed time of operation for each pump is also recorded and so a method for calculating pump flow is available. By determining the pump capacity from the manufacturer's characteristic curve, based on the elevation in the wet well, and multiplying this capacity by the incremental time of operation, the total flow can be calculated for any time period.

A convenient means of checking flow rates was available since on some occasions, the storm pumps operate but the settling basin does not overflow. The volume of water in the basin was measured and checked against the pumped volume as calculated below. Three separate storms in which the basin did not overflow were used to check the accuracy of the method. The results are listed in Table VI.

TABLE VI

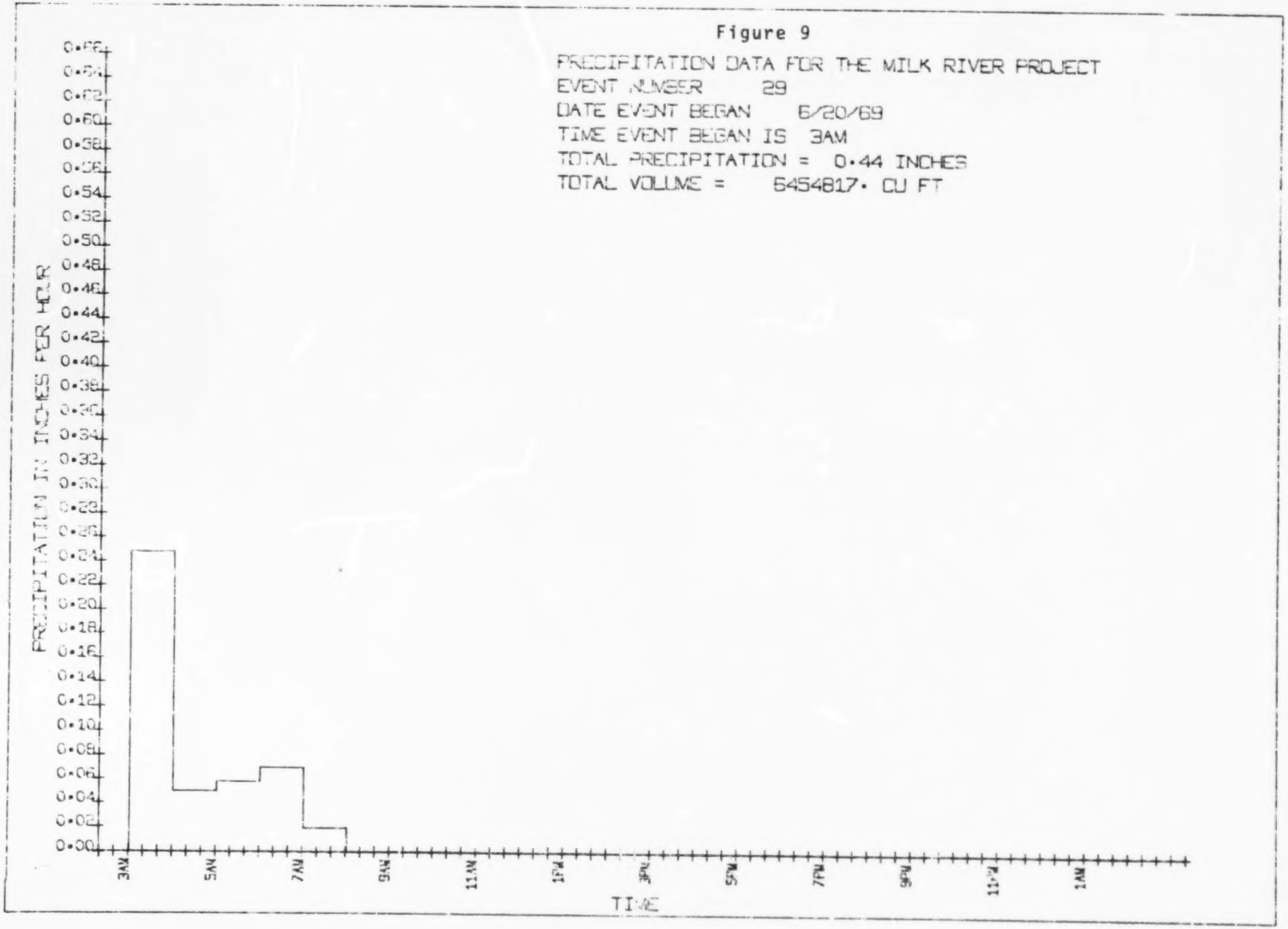
MEASUREMENT OF STORM VOLUMES

Date of Storm	Volume (cu ft) from Pumping Calculations	Volume (cu ft) From Basin Volume	% Difference
2-15-67	297,000	286,000	+3.7
3-22-67	515,000	495,000	+3.9
4-18-67	260,000	262,000	-0.8

Using the methods described above, rainfall and pumping hydrographs could then be constructed for any storm occurring in the Milk River Drainage basin. Data from all record storms for which quality data has been obtained have been processed in this manner.

Typical rainfall and runoff hydrographs are shown in Figures 9 and 10. Note the intermittent pumping sequence in Figure 10. The rainfall and pumping mass diagrams are shown in Figure 11; the accumulative rainfall and pumping data are shown in Tables VII and VIII. This is characteristic of most st m pump operations at the station and indicates that the m imum

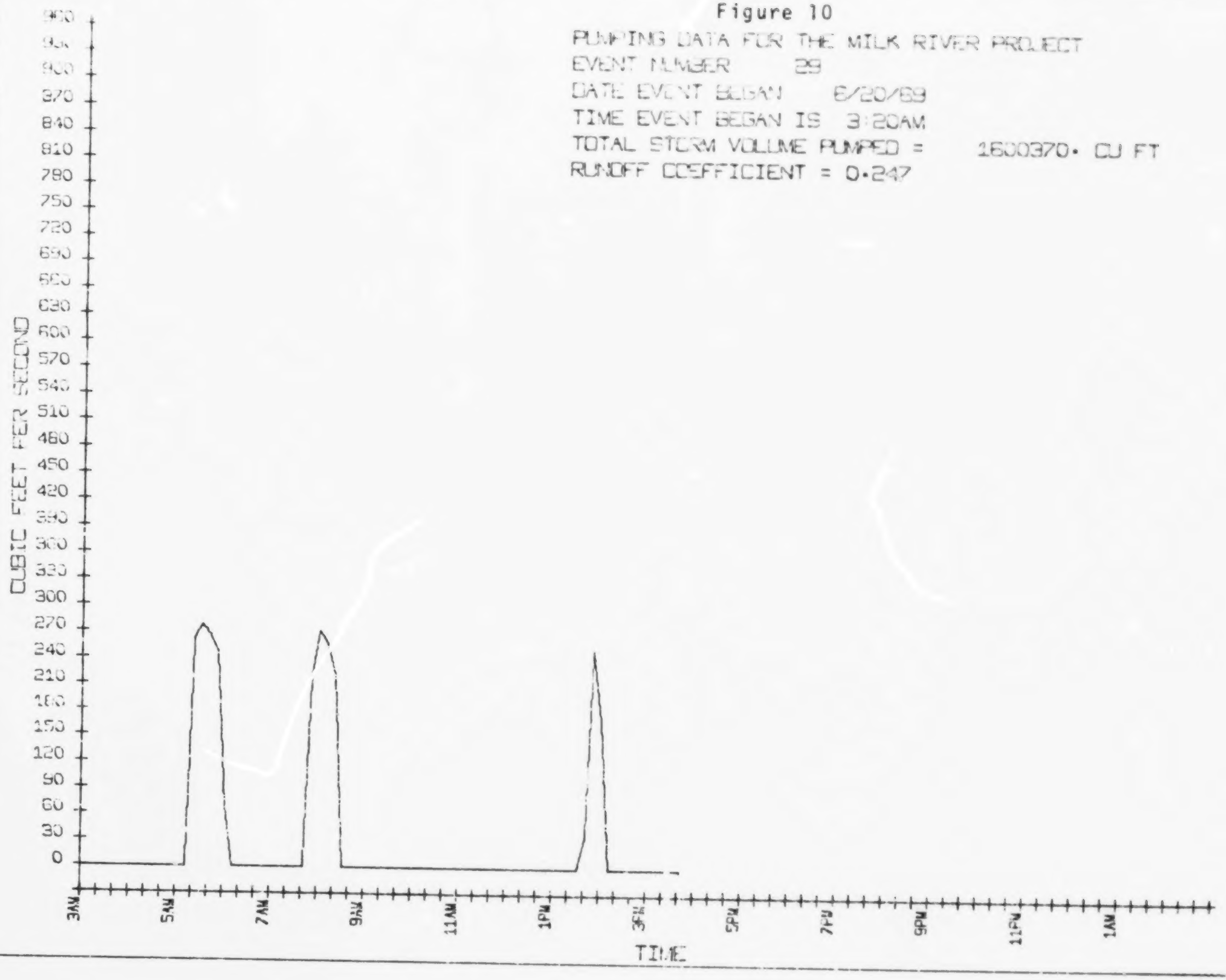
Figure 9



PRECIPITATION DATA FOR THE MILK RIVER PROJECT
EVENT NUMBER 29
DATE EVENT BEGAN 6/20/69
TIME EVENT BEGAN IS 3AM
TOTAL PRECIPITATION = 0.44 INCHES
TOTAL VOLUME = 6454817. CU FT

Figure 10

PUMPING DATA FOR THE MILK RIVER PROJECT
EVENT NUMBER 29
DATE EVENT BEGAN 6/20/69
TIME EVENT BEGAN IS 3:20AM
TOTAL STORM VOLUME PUMPED = 1600370. CU FT
RUNOFF COEFFICIENT = 0.247



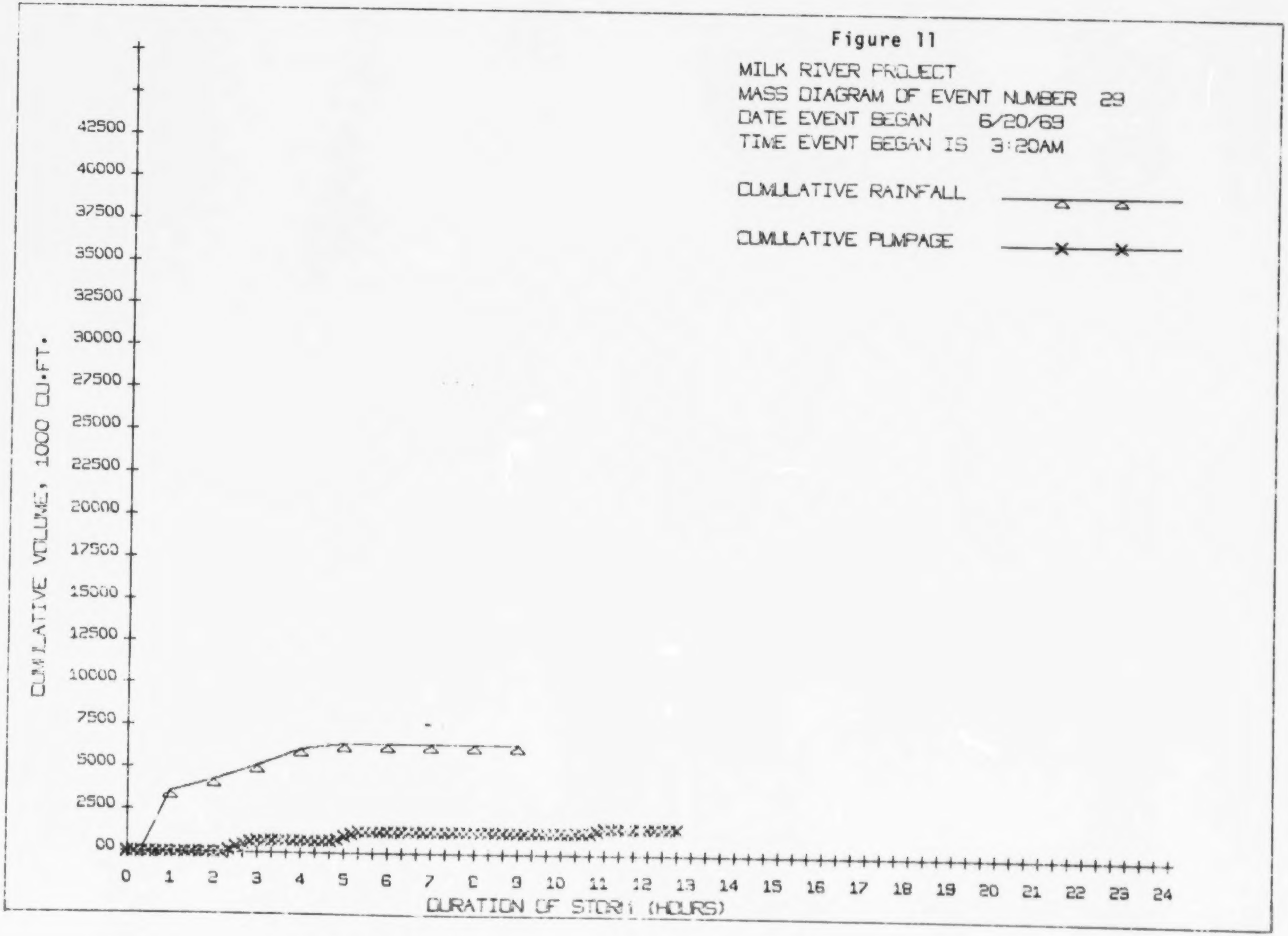


Figure 11

MILK RIVER PROJECT
MASS DIAGRAM OF EVENT NUMBER 29
DATE EVENT BEGAN 6/20/69
TIME EVENT BEGAN IS 3:20AM

TABLE VII

PRECIPITATION DATA FOR THE MILK RIVER PROJECT
EVENT NUMBER 29
DATE EVENT BEGAN 6/20/69

TIME	PRECIPITATION OVER AREA FOR HOUR ENDING	CUMULATIVE PRECIPITATION OVER AREA	VOLUME OF WATER FOR THE STORM	CUMULATIVE VOLUME FOR STORM
	INCHES/HOUR	INCHES	1000 CU FT	1000 CU FT
3AM	0.247	0.247	3594.	3594.
4AM	0.049	0.297	724.	4319.
5AM	0.057	0.355	839.	5159.
6AM	0.069	0.425	1012.	6172.
7AM	0.019	0.445	282.	6454.
8AM	0.000	0.445	0.	6454.
9AM	0.000	0.445	0.	6454.
10AM	0.000	0.445	0.	6454.
11AM	0.000	0.445	0.	6454.

EVENT TOTALS

TOTAL PRECIPITATION = 0.44 INCHES
TOTAL VOLUME = 6454817. CU FT

TABLE VIII

PUMPING DATA FOR THE MILK RIVER PROJECT
EVENT NUMBER 29
DATE EVENT BEGAN 6/20/69
TIME EVENT BEGAN IS 3 20AM AND IS PRINTED OUT IN TEN MINUTE INTERVALS
AVERAGE DRY FLOW FOR 10 MINUTE PERIOD IS 1650 CU FT

TIME	TOTAL VOLUME FOR TEN MINUTE PERIOD 1000 CU FT	CUMULATIVE STORM VOLUME 1000 CU FT	FLOW CFS
3.10AM	0.00	-0.83	0.00
3.20AM	0.00	-0.83	0.00
3.30AM	0.00	-0.83	0.00
3.40AM	0.00	-0.83	0.00
3.50AM	0.00	-0.83	0.00
4. 0AM	0.00	-0.83	0.00
4.10AM	0.00	-0.83	0.00
4.20AM	0.00	-0.83	0.00
4.30AM	0.00	-0.83	0.00
4.40AM	0.00	-0.83	0.00
4.50AM	0.00	-0.83	0.00
5. 0AM	0.00	-0.83	0.00
5.10AM	0.00	-0.83	0.00
5.20AM	0.00	-0.83	0.00
5.30AM	158.70	157.87	264.51
5.40AM	168.58	326.46	280.97
5.50AM	161.47	487.93	269.12
6. 0AM	149.62	637.56	249.37
6.10AM	42.11	679.67	70.19
6.20AM	0.00	679.67	0.00
6.30AM	0.00	679.67	0.00
6.40AM	0.00	679.67	0.00
6.50AM	0.00	679.67	0.00
7. 0AM	0.00	679.67	0.00
7.10AM	0.00	679.67	0.00
7.20AM	0.00	679.67	0.00
7.30AM	0.00	679.67	0.00
7.40AM	0.00	679.67	0.00
7.50AM	0.00	679.67	0.00
8. 0AM	133.18	812.86	221.97
8.10AM	164.84	977.70	274.74
8.20AM	156.53	1134.24	260.89
8.30AM	134.86	1269.10	224.76
8.40AM	0.00	1269.10	0.00
8.50AM	0.00	1269.10	0.00
9. 0AM	0.00	1269.10	0.00
9.10AM	0.00	1269.10	0.00
9.20AM	0.00	1269.10	0.00
9.30AM	0.00	1269.10	0.00
9.40AM	0.00	1269.10	0.00
9.50AM	0.00	1269.10	0.00

NOT REPRODUCIBLE

TABLE VIII (Cont.)

```
EVENT NUMBER                         29

 TIME      TOTAL VOLUME        CUMULATIVE          FLOW
            FOR   TEN          STORM
           MINUTE PERIOD       VOLUME

            1000 CU FT         1000 CU FT          CFS

10. 0AM        0.00            1269.10             0.00
10.10AM        0.00            1269.10             0.00
10.20AM        0.00            1269.10             0.00
10.30AM        0.00            1269.10             0.00
10.40AM        0.00            1269.10             0.00
10.50AM        0.00            1269.10             0.00
11. 0AM        0.00            1269.10             0.00
11.10AM        0.00            1269.10             0.00
11.20AM        0.00            1269.10             0.00
11.30AM        0.00            1269.10             0.00
11.40AM        0.00            1269.10             0.00
11.50AM        0.00            1269.10             0.00
12. 0AM        0.00            1269.10             0.00
12.10PM        0.00            1269.10             0.00
12.20PM        0.00            1269.10             0.00
12.30PM        0.00            1269.10             0.00
12.40PM        0.00            1269.10             0.00
12.50PM        0.00            1269.10             0.00
 1. 0PM        0.00            1269.10             0.00
 1.10PM        0.00            1269.10             0.00
 1.20PM        0.00            1269.10             0.00
 1.30PM        0.00            1269.10             0.00
 1.40PM        0.00            1269.10             0.00
 1.50PM       20.94            1290.04            34.90
 2. 0PM      152.29            1442.34           253.82
 2.10PM      104.70            1547.05           174.51
 2.20PM        0.00            1547.05             0.00
 2.30PM        0.00            1547.05             0.00
 2.40PM        0.00            1547.05             0.00
 2.50PM        0.00            1547.05             0.00
 3. 0PM        0.00            1547.05             0.00
 3.10PM        0.00            1547.05             0.00
 3.20PM        0.00            1547.05             0.00
 3.30PM        0.00            1547.05             0.00
 3.40PM        0.00            1547.05             0.00
 3.50PM        0.00            1547.05             0.00

EVENT TOTALS

TOTAL STORM VOLUME =      1600370.    CU FT
```

sized pump at Milk River has a rate which draws down the sewer storage at a rate greater than the sewer intrusion rate for most storms. This characteristic results in a pulsed or intermittent flow into the settling basin.

By averaging the area under each pulse of the curve in Figure 10 and extending the time-rate function until a continuous curve is developed, an estimate of the average influent flow rate can be made. The long-term average for Storm No. 29, for example, is 51 cfs (22,900 gpm), significantly less than the existing minimum capacity of 315 cfs (141,000 gpm). A considerable improvement in pumping efficiency could be achieved by containing short-term, high-rate peak flows in upstream storage while maintaining low average pumping rates at the Station.

ANALYSIS OF COMBINED SEWER OVERFLOW RATES

Because of the nature of combined sewer overflows, it is necessary to measure the magnitudes and frequencies of occurrence of both overflow rates and volumes. The problems associated with hydraulic loading, sludge production, and chemical feeding can then be assessed and an efficient, economical and workable design effected. The influent pollutional load and its final impact on the receiving water can then be estimated.

An analysis of records at the MRPS for the period 1960 through 1968 is contained in Table IX. The percent of time a selected flow rate was equal to or less than a given value was calculated. The overflow rate did not exceed the capacity of one large pump (410 cfs; 184,000 gpm) for almost 90 percent of the occurrences. A rate of 3 x 410 cfs only occurred an additional 8 percent of the time. The capital cost would be almost tripled over that to handle 410 cfs if treatment capacity were provided for 1230 cfs (551,000 gpm). The corresponding increase in operating time would be approximately 8 percent. More significantly, the total volume treated would be increased by about 15 percent.

The logarithmic distribution of flow rates is shown in Figure 12. The percent of time that the flow rate equaled or exceeded any given value can be extrapolated from this plot. The geometric mean overflow rate is extrapolated as that rate of pumping which would not be exceeded for 50 percent of all occurrences. The mean overflow rate of 187 cfs (83,900 gpm) is less than one-half the capacity of one large pump (410 cfs) in the existing facility. Even one small pump (305 cfs) is too large to handle the majority of storm overflow rates without periodic storage in the wet well. This fact was also apparent from the

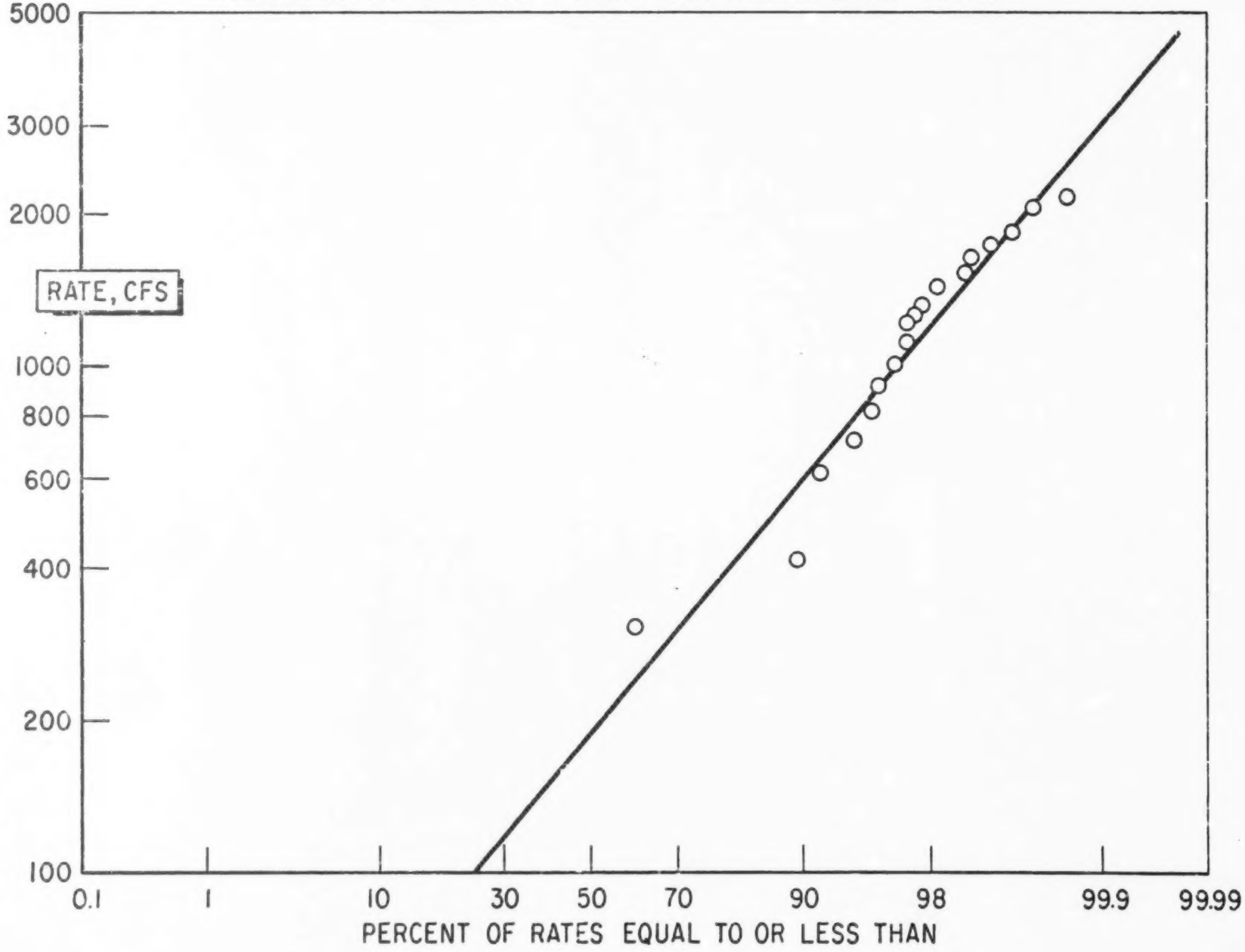
TABLE IX

DISTRIBUTION OF STORM OVERFLOW RATES (1960-1968)

Pump Combination	Rate (cfs)	Duration (min)	Accumulative Duration (min)	% Time	Total Volume (cu ft)	Accumulative Volume (cu ft)	% Volume at Rate Shown
1/0	305	20,955	20,955	60.09	383,476,500	383,476,500	44.76
0/1	410	10,237	31,192	89.45	251,830,200	635,306,700	74.16
2/0	610	826	32,018	91.82	30,231,600	665,538,300	77.68
1/1	715	889	32,907	94.37	38,138,100	703,676,400	82.14
0/2	820	405	33,312	95.53	19,926,000	723,602,400	84.46
3/0	915	133	33,445	95.91	7,301,700	730,904,100	85.31
2/1	1,020	275	33,720	96.70	16,830,000	747,734,100	87.28
1/2	1,125	126	33,846	97.06	8,505,000	756,239,100	88.27
4/0	1,220	29	33,875	97.14	2,122,800	758,361,900	88.52
0/3	1,230	101	33,976	97.43	7,453,800	765,815,700	89.39
3/1	1,325	136	34,112	97.82	10,812,000	776,627,700	90.65
2/2	1,430	125	34,237	98.18	10,725,000	787,352,700	91.90
1/3	1,535	225	34,462	98.82	20,722,500	808,075,200	94.32
4/1	1,630	16	34,478	98.87	1,564,800	809,640,000	94.51
3/2	1,735	127	34,605	99.23	13,220,700	822,860,700	96.05
2/3	1,840	85	34,690	99.48	9,384,000	832,244,700	97.14
4/2	2,040	53	34,743	99.36	6,487,200	838,731,900	97.90
3/3	2,155	58	34,801	99.80	7,499,400	846,231,300	98.78
4/3	2,450	71	34,872	100.00	10,437,000	856,668,300	100.00

Figure 12

DISTRIBUTION OF STORM OVERFLOW RATES (1960-1968)



pulsed operation. A large portion of the influent flow to Milk River could be treated under relatively steady-state conditions if pumping could be varied between 100 and 305 cfs. The total capacity of the sanitary pumping systems is only 11 cfs and is generally not available during times of storm.

This analysis is limited to the existing Milk River facility but several useful possibilities are suggested.

1. A similar correlation can be developed from rainfall distribution values corrected for runoff characteristics for design of new combined sewage treatment installations.
2. Optimum or most economical treatment or pumping analyses can be made on the basis of the most probable rainfall-runoff curves developed in 1.

ANALYSIS OF COMBINED SEWER OVERFLOW VOLUMES

It is also necessary for several reasons to evaluate variations in flow volumes in addition to flow rates. Firstly, the cost of retaining a given percentage of storms for discharge into the Detroit interceptor system during dry weather may be more economical than installing and operating treatment facilities, chemical inventory requirements, and to some extent, feed rate capacities which are a function of total storm volumes.

The distribution of storm overflow volumes for the years 1960 through 1968 is shown in Table X and plotted in Figure 13.

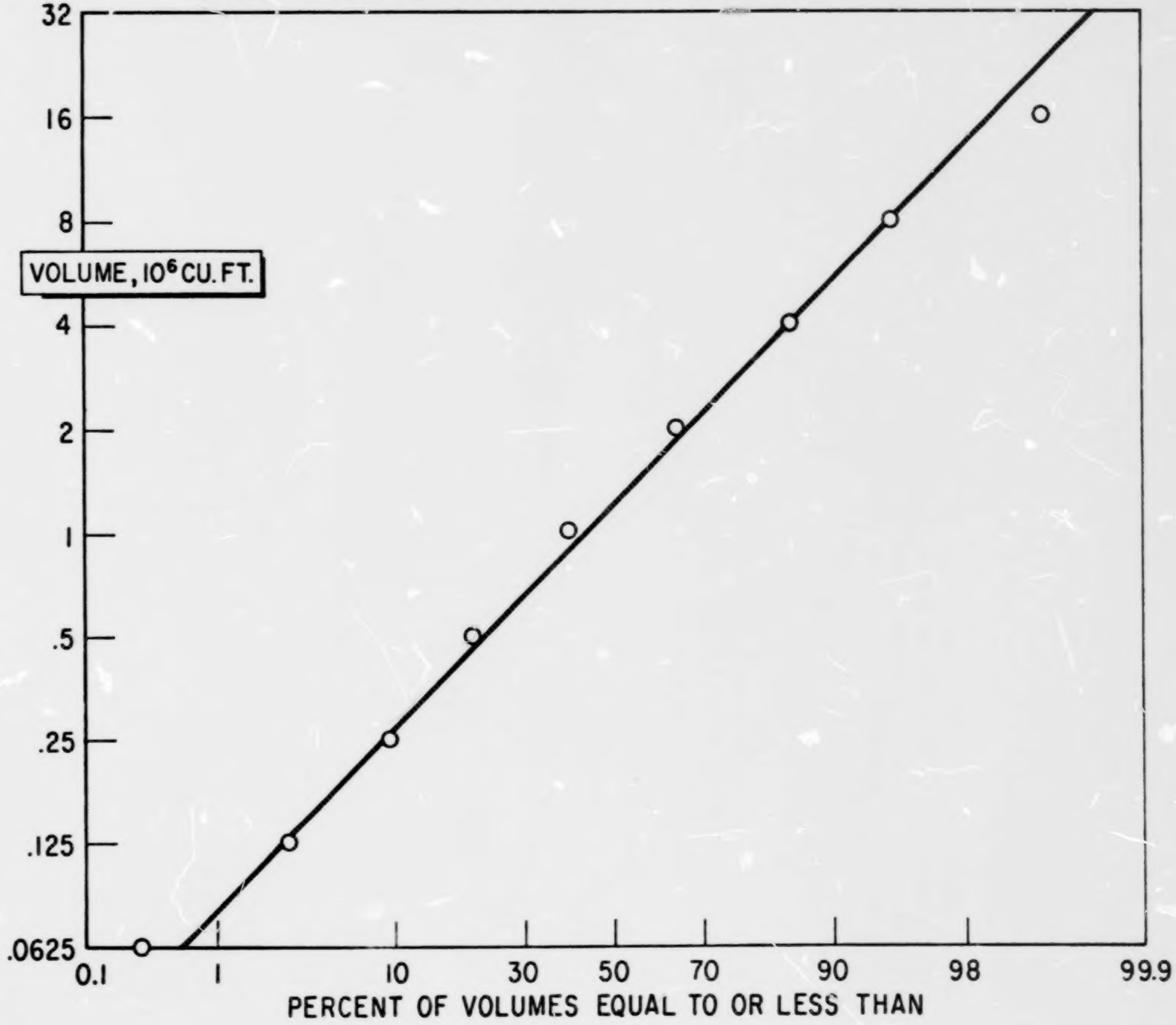
TABLE X

DISTRIBUTION OF STORM OVERFLOW VOLUMES (1960-1968)

Volume 10^6 Cu Ft	Volume 10^6 Gal	Total Occurrences	Cumulative Total	% Total/100
0- .06	0- .45	1	1	0.0030
.06- .12	.45- .90	9	10	.0296
.12- .25	.90- 1.87	23	33	.0976
.25- .50	1.87- 3.74	37	70	.2071
.50- 1.00	3.74- 7.48	64	134	.3964
1.00- 2.00	7.48- 15.96	83	217	.6420
2.00- 4.00	15.96- 31.92	71	288	.8521
4.00- 8.00	31.92- 63.84	33	321	.9497
8.00-16.00	63.84-127.68	15	336	.9941
16.00-32.00	127.68-255.36	2	338	1.0000

Figure 13

DISTRIBUTION OF STORM OVERFLOW VOLUMES (1960-1968)



The geometric mean pumped volume (50 percent of total occurrences) is approximately 1.2 million cu ft (8.98 million gal); 90 percent of the pumped volumes do not exceed 5.5 million cu ft (41.1 million gal).

FACTORS AFFECTING COMBINED SEWER OVERFLOWS

Another important aspect of the Milk River combined sewer overflow problem is that of understanding the factors which cause the overflow structure to operate.

From studies of the Pumping Station records, it has been estimated that the Milk River sewerage system has a storage capacity of approximately 600,000 cu ft (4,490,000 gal). The average dry weather flow for 1968 was estimated to be about 5 cfs (2,240 gpm). Under present operating practice, the maximum flow pumped to the sanitary interceptor before switching to storm operation is about 22 cfs (9,870 gpm).

By inserting these values in the conventional storage formula (Rate of Inflow - Rate of Outflow = Rate of Storage), the following relationship is derived.

Dry Weather Sanitary Flow and Storm Runoff - Sanitary Pumping = Rate of Storage

$$5 \text{ cfs} + CIA - 22 \text{ cfs} = \frac{600{,}000 \text{ cu ft}}{t \text{ (sec)}}$$

$$CIA = \frac{10{,}000}{t \text{ (min)}} + 17$$

Assuming an average runoff coefficient of 0.35 and a drainage basin area of 3990 acres, the intensity of rainfall required to cause an overflow becomes:

$$I = \frac{43}{t \text{ (min)}} + .012$$

Between 1960 and 1969, the median duration for all storms at Milk River was between three and four hours and 86 percent of the storms did not exceed eight hours. The minimum duration of importance would be the Time of Concentration or approximately 1.5 hours. Following is a tabulation of rainfall intensities required to cause overflow for a reasonable range of storm durations.

TABLE XI

THEORETICAL RAINFALL INTENSITY AND VOLUME TO CAUSE OVERFLOW

Storm Duration (Minutes)	Runoff (CIA) (cfs)	Storm Intensity (Inches/Hr)	Total Rainfall (Inches)
90	128	0.092	0.138
120	100	.072	.144
180	73	.052	.156
240	59	.042	.168
300	50	.036	.180
360	45	.032	.192
420	41	.029	.203
480	38	.027	.216

These calculations show the effect of storm duration both on the storm intensity required to cause overflow and on the accumulated rainfall required to cause overflow. From this analysis, it appears that the overflow at Milk River is more a function of total rainfall than of rainfall intensity. Surface storage and wetness factors would undoubtedly increase the total rainfall requirements shown in Table XI.

The number of days of pumping at Milk River compared with the number of days of various excess precipitations at a nearby gauging station are shown in Table XII. The data were not separated as to individual storms. Some error was introduced when storms or pumping runs extended into more than one day. The average number of days of pumping per year for the eight year period is about the same as the average number of days per year of precipitation slightly in excess of 0.2 inches. Considering surface storage and ground wetness, the data supports the rainfall-runoff analysis shown in Table XI.

Coupled with the rainfall-runoff distributions suggested earlier, the storage-runoff analysis can provide one rational approach to the analysis of combined sewer overflow systems.

TABLE XII

DAYS OF PUMPING AND VARIOUS EXCESS
PRECIPITATION AT STATION M-3

	Number of Days of Precipitation				No. Days of
Year	Measurable	<.1"	<.2"	<.3"	Storm Pumping
1961	116	77	52	40	61
1962	103	70	40	30	44
1963	87	54	30	25	23
1964	102	61	40	27	36
1965	127	74	53	42	45
1966	95	64	45	32	26
1967	102	72	49	34	41
1968	93	67	47	32	53
Average	78	67	45	32	41

INFLUENT-EFFLUENT QUALITY

The quality of the combined sewage should be evaluated before a complete statement of the Milk River problem can be outlined. This was necessary both for characterization of the influent and effluent sewage and for evaluation of the various chemical treatment systems. Under the terms of the Contract, characterization required 22 analyses on consecutive time-weighted samples from both the influent and effluent.

Laboratory Trailer

Laboratory facilities were not available at or near the Milk River site and had to be built. A mobile laboratory was designed and constructed for immediate use during the Contract and future use by the Federal Water Quality Administration at the conclusion of the Contract.

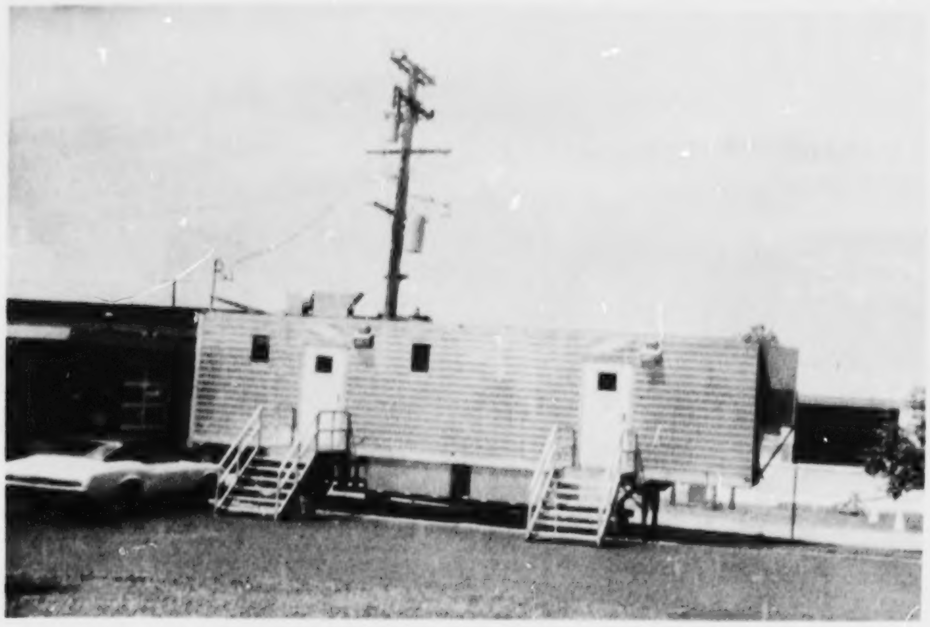
The space requirements were estimated on the basis of the expected analytical load during storm events, and consultation with FWQA personnel experienced in mobile laboratory design and operation. It was decided that the most practicable unit was a commercial semi-trailer. The solid construction of such units provides a stable platform without restricting mobility or load capacity.

The semi-trailer selected was 40-foot long, 8 feet wide, and 13 feet from ground to roof. It is the largest trailer presently allowed on the roads of most states without special permits. The interior capacity is approximately 2500 cu ft; it has almost 300 sq ft of floor space. The sides and roof are an aluminum skin over a steel frame. There are two doors on one side of the trailer and four windows (16" wide x 24" high) located at eye level. In addition to the normal "landing gear" on the front of the unit, a set of leveling jacks is provided behind the rear wheels of the trailer to give additional support and stability while the trailer is on site. Two sets of access steps and platforms with handrails are designed to be removed from the trailer and placed in storage boxes beneath the trailer.

An outside view of the completed trailer in place at the Milk River Pumping Station is shown in Figure 14.

Figure 14

Laboratory Trailer on Site at Milk River



The interior units of the laboratory were designed compactly because of the limited floor area available. The equipment required for similar analytical tests were grouped together in specific areas to reduce traffic problems. All of the equipment required for the bacteriological testing was grouped together at the front of the trailer; ovens and other equipment needed for solids determination were placed at the rear. Each location had a conveniently located sink. A centrally located air compressor and vacuum pump were connected to headers along three sides of the trailer.

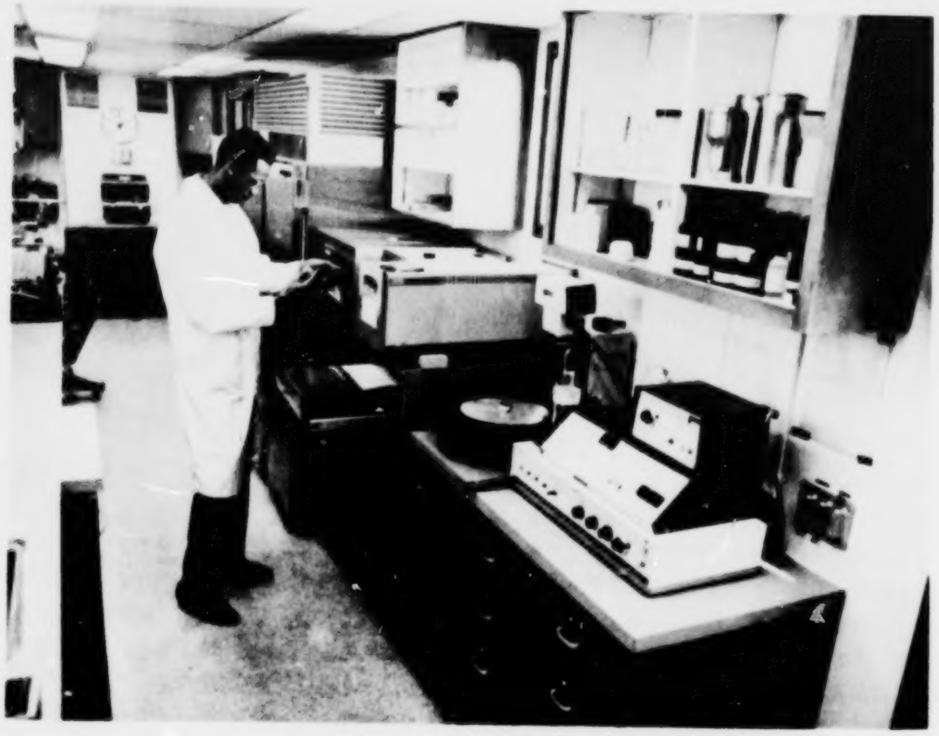
The design of the fume hood was a special problem. A negative pressure would develop in the trailer if ventilating air came from within, so a unit was obtained that allows a portion of the vent air to come from the outside. Special equipment was designed to heat this outside air in cold weather.

Conventional heating and air conditioning units of the type that are used with most trailers were not suitable for the laboratory. Heat from furnaces, hot plates and other equipment required a unit with a large, flexible cooling capacity. A custom-built heating-ventilating and air-conditioning (HVAC) unit designed to fit on the nose of the trailer was installed. A hood with ventilating fan was installed over the muffle furnace and drying ovens to reduce the load on the HVAC equipment as well as to carry off fumes and odors. The conditioned air is conveyed through ceiling ducts for the full length of the trailer and is distributed through continuous large area, low rate diffusers. Local heating or cooling can be controlled by adjustment of the diffusers.

A view of the interior looking toward the front of the trailer is shown in Figure 15; a plan view of the trailer interior is shown in Figure 16.

Figure 15

Interior of Laboratory Trailer



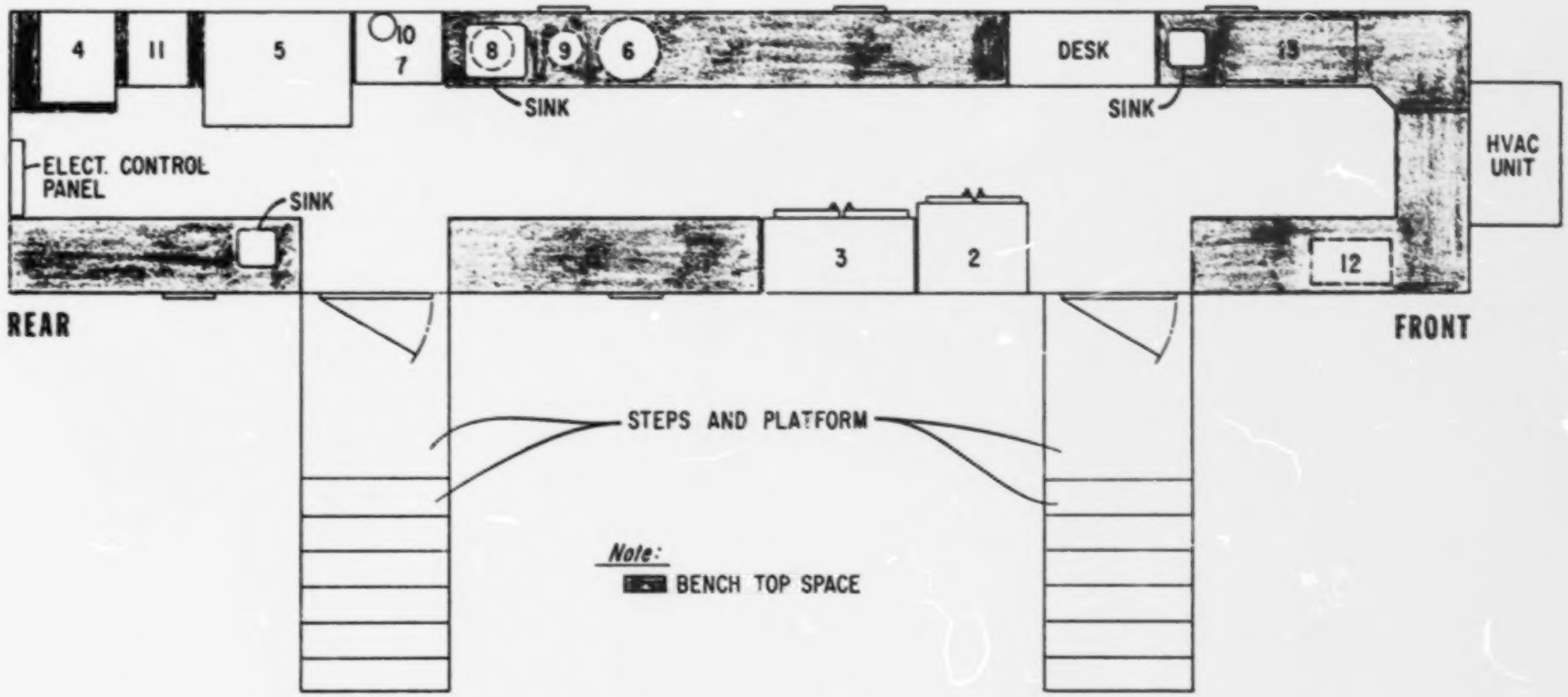
Sampling

One of the most difficult problems at Milk River was that of sampling. Flow rates varied from 305 to 2450 cfs (137,000 to 1,100,000 gpm). Influent sewage depths varied from 2 to 17 feet with no dry well available for positive head devices, and a representative effluent sample had to be obtained from an inaccessible weir approximately 210 feet in length.

Figure 16

MOBILE LABORATORY TRAILER

2. REFRIGERATOR
3. B.O.D. INCUBATOR
4. MUFFLE FURNACE
5. FUME HOOD
6. WATER HEATER (Under counter)
7. LABORATORY GLASSWARE WASHER
8. AIR COMPRESSOR (Under counter)
9. VACUUM PUMP (Under counter)
10. WATER STILL (On wall)
11. DRYING OVENS (2)
12. SMALL WATER BATH
13. LARGE WATER BATH


4
11
5
10
7
8
9
6
DESK
13
SINK
SINK
HVAC UNIT
ELECT. CONTROL PANEL
SINK
3
2
12
REAR
FRONT
STEPS AND PLATFORM
Note:
▬ BENCH TOP SPACE

FLOOR PLAN & GENERAL ARRANGEMENT

The basin effluent was sampled by four 1-inch vertical suction lines spaced evenly along the effluent weir. The suction lines drew a sample from points between the skimming baffle and effluent weir at a depth above the bottom of the skimming baffle and just below the outlet weir. The four sampling pipes fed into a common header through equal lengths of pipes using the same fittings to equalize the head loss in each line and permit samples which make up the composite to be of equal volume. Another line was run to the Milk River to permit sampling and calibration of automatic analytical equipment when the basin was empty.

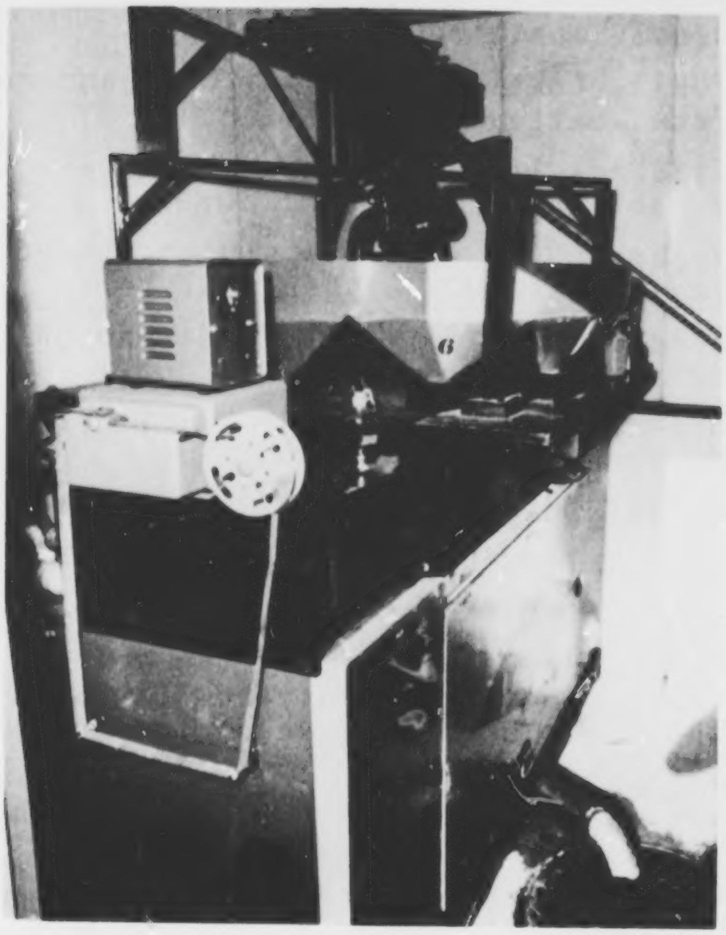
Influent samples were obtained by means of a submersible pump suspended in the wet well beyond the bar screens within the transition structure between sewer and wet well. All main sampling lines in both the influent and effluent systems were 2-inch in diameter and flowed constantly during the sampling period.

Because of the importance of sampling, automatic samplers were designed and constructed specifically for the work at Milk River. The samplers were designed to collect adjustable grab samples from the continuously flowing 2-inch pipe stream, composite them for variable periods and hold them in a refrigerated compartment for periods up to about three hours. The sampling program was controlled by a continuous punched tape program which varied the collection time of each composite, the number of grab samples in each composite, and each of the variables from one sampling time to another. The size of each grab sample was controlled externally. Construction of the effluent sampler can be seen in Figure 17. The influent sampler was identical except for the tape control unit and reader which was fabricated from basic components in contrast to the use of commercially available units as seen on the effluent sampler.

Analytical Methods

The following analyses were performed on the Milk River influent and effluent samples. The procedures as outlined in Standard Methods for the Examination of Water and Wastewater[2] were followed except for the modifications noted below. The order and abbreviations are identical to those used in Tables XIII and XIV.

Figure 17
Effluent Sampler



1. Date - calendar day of record.
2. Storm (STRM)
3. Sample (SMPL)
4. Time
5. pH - Continuously monitored and recorded.
6. Turbidity (TUR) - Continuously monitored and recorded using a surface scatter turbidimeter.
7. Temperature (TEM) - Influent temperature only was continuously monitored and recorded.
8. Dissolved oxygen (DO) - Continuously monitored and recorded using a dissolved oxygen analyzer.
9. Suspended solids (SUS SOL) - Filtered through a glass mat and dried at 103°C.
10. Volatile suspended solids (VOL SOL) - Filtered through a glass mat and fired at 600°C.
11. Total solids (TOT SOL) - Evaporated at 103°C.
12. Total volatiles solids (TOT VOL SOL) - Fired at 600°C.
13. Settleable solids (SET SOL) - By volume using an Imhoff cone.
14. Chlorides (TOT CL) - Determined by the Volhard method. The Volhard Method B as given in the 11th Edition of Standard Methods was used because of the expected high concentration of chloride ions.
15. Total hardness (TOT HARD) - EDTA titrametric method with methylthymol blue as the indicating agent
16. Calcium hardness (CA HARD) - EDTA titrametic method with methylthymol blue as the indicating agent.
17. Chlorine demand (CL DEM) - Excess chlorine was determined by amperometric titration.
18. Biochemical oxygen demand (BOD) - Winkler method with azide modification to remove interference from nitrites.
19. Nitrogen (TOT N) - Samples were returned to Midland for analysis. They were preserved by adding 40 mg/l Hg^{++} and frozen as recommended in a Report to U. S. Public Health Service[3]. The method of analysis used was the total Kjeldahl method for ammonia and organic nitrogen.
20. Total phosphates (TOT P) - Samples were returned to Midland for analysis by the stannous chloride method.
21. Oil and grease (OIL) - Samples were quick-frozen and returned to Midland for analysis by the Soxhlet Extraction Method

22. Inorganic carbon (INR C) - Infrared carbonaceous analyzer.

23. Organic carbon (ORG C) - Infrared carbonaceous analyzer.

24. Total coliforms (TOT COLI) - Membrane filtration and incubation at 35°C for 18-24 hours on Bacto-m-Endo broth MF.

25. Fecal coliforms (FEC COLI) - Membrane filtration and incubation at 44.5°C for 24 hours on Bacto-m-FC broth.

26. Fecal Streptococci (FEC STRP) - Membrane filtration and incubation at 35°C for 48 hours on Bacto-m-Enterococcus agar.

27. Storm volume (STRM VOL) - Calculated from station records (corrected for changes in elevation in wet well).*

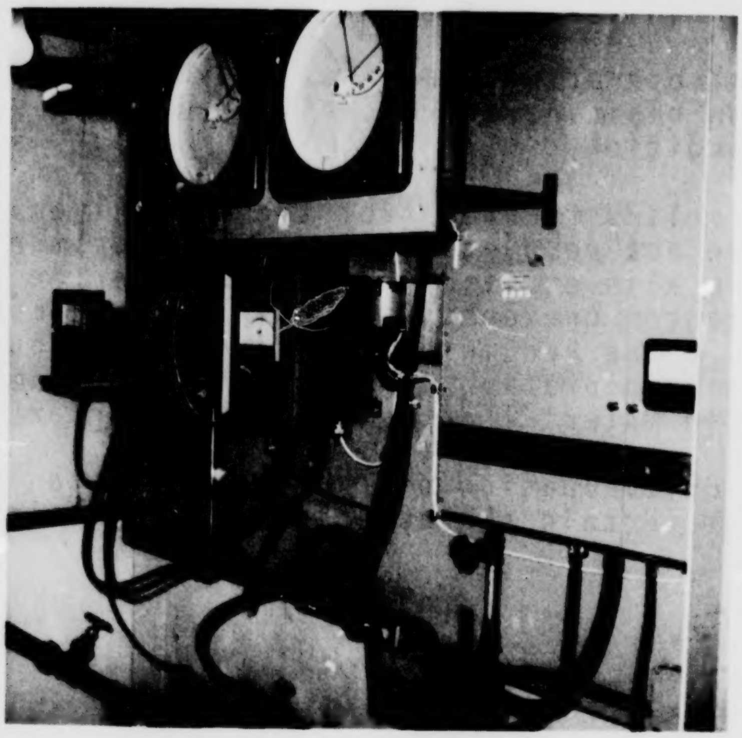
28. Conductivity (not tabulated) - Influent stream only was continuously monitored and recorded.

The analyses listed above were performed within three hours of the time of sample collection except for those which were performed in Midland on preserved samples.

Calibration of the automatic pH and dissolved oxygen recorders was extremely difficult to maintain under the conditions of intermittent operation and variation in solids content of the Milk River combined sewage. The surface scatter turbidimeter performed reliably but was subject to plugging by large solids. The automatic analytical equipment in the effluent sampler building is shown in Figure 18.

Figure 18

Automatic Analytical Equipment



Analytical Results

Results of the analyses of influent and effluent samples from the Milk River retention basin are summarized in Tables XIII and XIV. Each sample is a composite of one minute grab samples collected continuously during a pumping sequence. Several factors affect the nature of these data and should be considered in quantitative evaluation. Some of the more important factors are:

1. Intermittent operation of the pumping station causes widely varying sewer velocities, unstable vertical concentration gradients, and poor mixing in the influent combined sewage. Since the flow depth in the sewer varied from 2 to 17 feet, the optimum influent sample point was necessarily selected on the basis of empirical testing and weighted judgment.

2. The effluent sample points were more ideally located but other factors led to difficulties in interpreting the effluent quality data. Specifically these factors were:

   a. Non-uniform distribution of flow across the effluent weir. Each pump or pump combination had a unique flow distribution pattern across the weir.

   b. Circulating flows were created within the basin during many pumping sequences as a result of flow diversion by the effluent weir.

   c. Effective detention times in the basin varied considerably depending on the pumping configuration.

   d. In normal operating practice, the Milk River retention basin never reaches a hydraulic steady-state condition.

   e. Settled solids flushed from the Milk River retention basin are not removed during the winter months. This condition also existed whenever weather or other circumstances prevented flushing of the basin between events. These accumulated solids were scoured during many operations causing effluent samples to contain material deposited during prior storms.

For the reasons noted, evaluation of the quality data is limited to a study of certain characteristics of the influent sewage.

TABLE XIII

MILK RIVER PROJECT - SUMMARY OF INFLUENT WATER QUALITY

DATE	STRM	SMPL	TIME	PH	TUR	TEM	DO	SUS SOL	VOL SUS SOL	TOT SOL	TOT VOL SOL	SET SOL	TOT CL	TOT HARD	CA HARD	CL DEM	BOD	TOT N	TOT P	OIL	INR C	ORG C	TOT COLI	FEC COLI	FEC STRP	CUM STORM VOLUME	STORM	SMPL
					JTU	°F	MG/L																NO./100ML X 10^6			10^3 FT^3		
10 3 68	13	01	2300		54			247	153			5	78	72	72	10.1	72	4.8	2.3	21.0	11	89	44	24.0	.290	1473.77	13	01
10 18 68	14	01	1612	6.7	54	66	3.2	72	27	497	171	1	55	139	142	6.8	95	6.6	1.8	21.5			19	7.9	.650	103.96	14	01
10 18 68	14	02	1712	7.2	19	66	5.5	74	38	487	173	2	68		90	6.8	89	3.4	1.3	19.5			9	0.1	.060	589.11	14	02
10 18 68	14	03	1812	7.2	19	66	5.0	141	81	604	264	8	62	94		8.8	121	4.9	1.5	38.1			30	0.8	.570	657.93	14	03
10 18 68	14	04	1912	7.7	19	66	4.3	6	2	281	108	0	43	106	74	7.0	34	4.3	1.6	7.3			2	0.2	.026	657.93	14	04
10 18 68	14	05	2012	7.2	19	66	4.3	26	12	277	94	0	44	107	78	6.6	31	2.8	1.5	7.0			3	0.5	.029	657.93	14	05
11 15 68	15	01	0530			45		464	314	721	428	15	36	182	106	10.9	216	3.1	1.1	76.0		256	14	1.0	.640	251.44	15	01
11 15 68	15	02	0550			45		206	160	400	230	10	25	56	54	11.1	99	1.5	1.1	25.0		284	4	0.6	.190	699.05	15	02
11 15 68	15	03	1350			45		57	32	285	112	2	39	75	69	9.3	120	2.7	1.6	2.2		67	1	0.6	.079	1690.37	15	03
11 16 68	16	01	1935					32	19	261	85	0	32	94	87	8.9	23	2.8	1.8	2.9	7	27	18	3.0	.200	531.85	16	01
11 16 68	16	02	2020					43	22	243	89	1	32	89	74	10.8	23	3.0	1.4	7.2	12	24	28	4.9	.058	531.85	16	02
11 16 68	16	03	2035					85	36	230	94	2	21	86	58	11.8	32	2.8	1.3	6.0	11	32	24	4.0	.290	758.36	16	03
11 16 68	16	04	2118					50	28	240	100	2	21	92	71	12.2	33	2.8	1.3	11.7	16	27	17	1.2	.102	1150.16	16	04
12 13 68	17	01	0305	7.5		48	7.8	766	564	1450	611	50	263	179	117	10.2	360	5.0	1.0	28.0			9	0.7	.760	66.11	17	01
12 13 68	17	02	0345			48	7.6	116	88	385	139	6	73	148	114	6.2	105	5.0	1.4	25.0			2	0.4	.490	346.70	17	02
12 13 68	17	03	0445	6.2		48	7.5	60	24	361	85	1	103	224	109	5.3	52	5.0	1.2	4.4			1	0.4	.140	465.01	17	03
12 19 68	18	01	1130	8.0	21	48	3.8	95	56	1401	209	3	707		109		93	5.7	1.4	27.5			7	0.8	.560	829.57	18	01
12 27 68	19	01	1830					107	43	3685	136	0	335	169	134		60	5.3	1.3	14.0			3	0.8	.270	1143.54	19	01
01 17 69	20	01	1910					466	284	3056	403	21	1372	232	186	8.5	376	7.6	1.0	99.9	18	262	10	1.6	.640	0.00	20	01
01 17 69	20	02	2025					137	58	1956	192	2	923	136	132	6.9	93	4.2	0.9	22.5	10	62	1	0.5	.370	672.76	20	02
01 17 69	20	03	2125					261	94	919	161	2	348	142	104	10.0	60	2.0	0.8	10.0	8	33	1	0.3	.113	743.49	20	03
01 17 69	20	04	2310					96	64	770	163	0	305	189	93	11.1	47	10.2	0.3	11.1	5	49	0	0.0	.003	1826.84	20	04
01 18 69	20	05	0730					102	59	558	127	2	216	148	111	9.9	44	2.3	0.8	11.0	6	31	0	0.0	.009	3239.95	20	05
01 18 69	20	06	0600					5	5	528	115	0	206	165	114	11.5	49	2.0	0.6	6.7	7	20	0	0.1	.027	3831.07	20	06
01 18 69	20	07	1135					24	22	522	142	0	174	187	99	8.9	50	2.3	1.6	10.0	11	26	1	0.8	.144	5272.29	20	07
01 18 69	20	08	1440					87	57	585	165	4	163	208	136	10.3	99	1.5	2.7	10.6	11	49	1	.3	.068	5853.09	20	08
01 18 69	20	09	1800					46	46	570	164	0	160	233	160	10.1	96	2.6	2.9	11.5	15	36	0	.3	.044	6264.90	20	09
03 24 69	21	01	1125	7.1	47	47	3.2	138	77	1296	231	1	323	240	232	7.7	101	7.5	1.4	27.9	29	84	6	1.5	.650	0.00	21	01
03 24 69	21	02	1200	7.4	47	46	5.1	139	86	1272	212	1	303	260	212	7.5	100	9.2	3.1	4.4	29	78	3	.7	.480	0.00	21	02
03 24 69	21	03	1222	7.2	54	46	5.6	216	125	604	157	3	163	144	112	5.0	97	4.1	2.3	33.6	13	75	1	.4	.250	201.40	21	03
03 24 69	21	04	1322	7.2	63	47	5.6	107	42	503	100	1	154	112	312	5.1	42	3.6	2.5	13.5	14	44	1	.1	.100	639.15	21	04
03 24 69	21	05	1422	7.4	47	46	5.6	119	42	504	118	3	138	112	116	4.3	48	5.1	2.4	10.1	13	39	1	.1	.114	639.15	21	05
03 25 69	21	06	1000	7.6	27	50	4.7	24	21	598	176	1			192	4.5	35	6.3	2.0	11.4	27	31	3	.4	.230	6714.65	21	06

TABLE XIII

MILK RIVER PROJECT - SUMMARY OF INFLUENT WATER QUALITY (CONTINUED)

DATE	STRM	SMPL	TIME	PH	TUR	TEM	DO	SUS SOL	VOL SUS SOL	TOT SOL	TOT VOL SOL	SET SOL	TOT CL	TOT HARD	CA HARD	CL DEM	BOD	TOT N	TOT P	OIL	INR C	ORG C	TOT COLI	FEC COLI	FEC STRP	CUM STORM VOLUME	STORM	SMPL
					JTU	°F								MG/L									NO./100ML X 10^6			10^3 FT^3		
04 2 69	22	01	0300	8.1			4.7	179		1422	361	4	284	280	199		93	5.8	1.0	23.0	45	51				442.59	22	01
04 2 69	22	02	0400	7.8			5.3	267		720	364	5	220	130	121		92	2.3	1.1	26.0	22	69				703.81	22	02
04 2 69	22	03	0500	7.1			5.8	188		560	132	2	170	111	98		50	1.6	0.7	12.3	16	44				1254.35	22	03
04 2 69	22	04	0600	7.0			5.7	145		420	230	2	96	105	95		26	2.1	0.5	2.2	16	27				1254.35	22	04
04 2 69	22	05	0800	7.1				69		368	168	1	82	140	133		21	3.2	0.5	2.8	14	24				1551.72	22	05
04 17 69	23	01	1348					258	162	674	204	3	131	192	137	9.0	128	7.3	2.1	33.4	21	54	3	1.5	.990	35.93	23	01
04 17 69	23	02	2010					176	98	769	174	2	124	179	141	8.4	83	8.5	2.2	25.2	19	83	4	1.1	.870	579.95	23	02
04 17 69	23	03	2103					216	104	556	159	2	89	169	118	8.6	34	5.6	1.3	9.3	27	45	2	1.0	.160	850.46	23	03
04 17 69	23	04	2156					72	66	432	156		57	216	164	8.9	17	3.8	1.2	0.4	26	42	1	.5	.140	12034.89	23	04
04 21 69	24	01	1330	7.5	46	40	5.4	57	57	802	250	1	145	386	254	7.3	53	13.0	3.7	12.9	4	32	8	5.0	.360	0.00	24	01
04 21 69	24	02	1630	7.5	37	40	5.5	76	76	589	173	0	124	244	161	6.9	36	9.9	3.0	6.8	11	36	2	1.4	.220	548.21	24	02
04 21 69	24	03	1830	7.5	34	40	5.0	52	35	536	160	0	113	223	181	6.4	38	9.0	2.6	5.4	13	26	2	1.0	.069	935.60	24	03
05 18 69	25	01	0714	7.4	55	61	2.5	190	178	541	211	12	110	137	100	4.8	56	4.5	1.2	12.1	17	86	5	1.0	.270	103.66	25	01
05 18 69	25	02	0916	7.5	32	61	4.6	24	20	216	85	3	39	83	69	7.3	27	2.9	0.8	3.4	2	23	1	.1	.072	659.30	25	02
05 18 69	25	03	1053	7.3	34	59	4.3	20	35	202	67	1	36	70	45	10.1	24	2.7	0.8	2.8	2	17	1	.1	.059	1122.30	25	03
05 18 69	25	04	1212	7.3	36	59	4.9	40	30	204	79	2	32	79	57	9.2	43	3.4	1.0	5.6	2	21	1	.1	.071	6152.47	25	04
06 1 69	26	01	0630						412	149	532	16	8 4	64	165	122			1.1	21.0	20	80				4003.61	26	01
06 5 69	27	01	0340	7.4	31	38	4.8	114	87	528	364	4	85	168	123	7.0	62	9.1	1.1	18.6	14	76	3	1.5	.460	462.37	27	01
06 5 69	27	02	0620	7.4	29	38	5.6	44	37	342	200	1	67	134	97	6.5	44	4.8	0.9	7.0	6	32	2	1.4	.250	436.72	27	02
06 8 69	28	01	1018	7.4			3.6	119	89	318	186	6	56	135	103	4.7	48	6.5	1.3	17.4	12	67	4	2.4	.067	28.49	28	01
06 8 69	28	02	1448	7.6			4.0	70	37	258	199	3	53	117	103	5.2	31	6.3	1.8	12.6	12	42	2	1.1	.160	1134.55	28	02
06 8 69	28	03	1730	7.7			4.2	78	36	237	157	2	46	136	124	8.3	18	3.9	1.3	11.6	17	33	3	1.6	.180	2177.62	28	03
06 20 69	29	01	0521	7.4	78	47	6.3	367	128	551	193	4	57	117	82	5.4	71	4.6	0.6	17.1	6	62	2	0.6	.410	0.00	29	01
06 20 69	29	02	0752	7.3	40	47	8.4	98	43	218	86	1	28	82	67	9.7	29	2.6	0.7	8.2	2	29	2	0.6	.003	679.67	29	02
06 20 69	29	03	1349		38			53	37	275	96	2	44	109	107	3.5	48		0.4	13.8	7	44	2	1.9	.170	1630.37	29	03
06 23 69	30	01	1207	7.4		64	6.9	183	85	332	113	4	39	117	110	3.9	54	5.4	2.7	13.5	17	47	4	1.8	.490	436.95	30	01
06 30 69	31	01		7.5		48	5.7	121	64	347	180	1	28	136	105	5.9		5.0	0.9	4.0	12	33				1300.1[illegible]	31	01
07 11 69	32	01	1810	7.1			4.6	364	166	525	205	6	57	141	111	6.3	100		1.2	19.5	8	94	6	4.2	.770	940.31	32	01
07 17 69	33	01	1210				5.1	233	97	502	169	5	28	92	85	7.2	45	4.1	.7	5.6	0	46	8	5.4	.280	1423.24	33	01
07 17 69	33	02	1515				5.0	169	65	410	144	2	28	131	117	6.7	40	5.1	1.4	6.7	8	32	4	3.6	.210	4071.61	33	02

TABLE XIV

MILK RIVER PROJECT - SUMMARY OF EFFLUENT WATER QUALITY

DATE	STRM	SMPL	TIME	PH	TUR JTU	TEM °F	DO MG/L	SUS SOL	VOL SUS SOL	TOT SOL	TOT VOL SOL	SET SOL	TOT CL	TOT HARD	CA HARD	CL DEM	BOD	TOT N	TOT P	OIL	INR C	ORG C	TOT COLI NO./100ML X 10^6	FEC COLI	FEC STRP	CUM STORM VOLUME 10^3 FT^3	STRM	SMPL
07 22 68	09	01	2228					248	155	492	160	1	50		84	7.6	126	6.3	4.8	31.0			2	0.6	.288	500.79	09	01
07 22 68	09	03	2247					250	162	474	142	5	40			6.2	123	4.8	3.9	21.0			1	1.4	.313	983.07	09	03
07 22 68	09	04	2302					188	144	367	126	2	41		90	5.9	52	5.5	3.6	18.0			9	1.6	.216	1031.75	09	04
07 22 68	09	05	2317					94	95	322	114	1	40	206	98	4.8	55	3.6	4.2	13.0			3	0.5	.099	1031.75	09	05
07 22 68	09	06	2347					108	111	370	110	1	37	125	91	7.5	38	3.4	3.2	18.0				2.1	.134	1031.75	09	06
07 23 68	09	07	0017					62	72	281		0	40	104	93	8.6	30	3.4					1	0.5	.073	1031.75	09	07
07 23 68	09	08	0047					102	87	244	51	5	25	82	67	9.3	25	2.8					0	1.4	.006	1031.75	09	08
07 23 68	09	09	0950					133	90	252	77	1	18	94	76	9.5	26	2.3					0	0.2	.003	2769.18	09	09
07 23 68	09	10	1500					110	74	285	102		11	98	69	9.6	24	3.1					1	0.2	.094	3731.68	09	10
08 16 68	11	01	2126					404	75	835	186	5	279	68	65	8.2	62	4.4	3.3	22.0			7	3.8	.001	1591.94	11	01
08 16 68	11	02	2141					207	33	475	80	1	272	54	48	6.9	84	1.7	3.0	28.0			1	0.2	.170	3490.40	11	02
08 16 68	11	03	2226					58	16	331	79	2	270	66	50	7.7	55	2.8	2.6	25.0			2	0.4	.115	7399.11	11	03
08 16 68	11	04	2326					38	3	305	80	1	287	77	55	5.3	34	2.1	2.9	27.0			3	0.5	.041	12280.64	11	04
08 17 68	11	05	0035					81	15	305	83	1	279	156	89	5.4	43	2.6	2.2	21.0			4	0.8	.052	15784.81	11	05
08 17 68	11	06	0400					46	14	384	136	1	270	190	132	6.1	32	2.0	2.8	19.0			0	0.1	.006	19005.67	11	06
08 19 68	12	01	1445					181	73	519	188	3	57	159	95	5.1	58	5.4	2.7	9.4			23	9.2	.150	121.46	12	01
08 19 68	12	02	1500					19	12	386	143	1	39	112	99	7.5	50	2.6	2.1	11.2			9	6.8	.062	550.57	12	02
08 19 68	12	03	1515					149	49	403	146	1	39	112	68	7.8	30	3.2	2.2	5.1			7	2.0	.042	1268.23	12	03
															84	7.6	31	3.4	2.2	6.2			1	0.8	.020	2310.81	12	04
10 03 68	13	01	2315	10.9	31		2.3	148	64	421	168	0	43	139	93	9.8	62	7.1	2.0	15.0	13	57	2	0.6	.330	910.46	13	01
10 03 68	13	02	2330	10.8	30		2.2	132	48	406	174	0	28	110	71	13.5	60	6.3	2.0	13.9	10	56	0	0.3	.080	910.46	13	02
10 04 68	13	03	0030	10.8	28		2.1	138	90	309	108	0	28	82	69	13.9	43	3.0	2.0	11.7	10	55	1	0.3	.010	1473.77	13	03
10 18 68	14	01	1619	7.4	63		1.5	116	50	507	208	4	44	112	190	6.9	93	4.6	1.0	30.0			8	1.7	.390	657.93	14	01
11 15 68	15	01	0545	7.5	48		6.1	170	128	360	184	4	32	83	75	9.6	103	2.3	1.0	56.8		147	4	0.8	.210	699.05	15	01
11 15 68	15	02	0600	7.5	47		6.1	290	132	393	195	4	25	98	60	9.8	105	2.5	1.0	33.3		114	6	0.8	.270	903.65	15	02
11 15 68	15	03	0615	7.5	44		6.0	220	102	373	174	4	32	76	54	9.3	101	2.7	1.1	25.5		101	5	0.6	.130	1003.50	15	03
11 15 68	15	04	1345	7.5	37		6.5	104	72	298	135	2	18	91	62	6.7	71	3.6	1.6	8.9		78	4	0.8	.134	1637.67	15	04
11 15 68	15	05	1400	7.5	25		6.4	136	90	346	173	1	25	96	67	8.1	76	3.7	1.5	13.0		92	4	8.5	.081	1690.37	15	05
11 16 68	16	01	2020		33		6.2	108	58	288	123	2	32	83	70	8.4	45	2.8	1.5	12.3	8	43	10	1.2	.180	501.65	16	01
11 16 68	16	02	2035		33		6.0	99	49	270	102	2	28	79	71	8.3	48	2.8	1.4	17.2	8	43	12	1.7	.210	673.61	16	02
11 16 68	16	03	2050		34		6.2	138	78	288	117	3	28	77	80	8.6	54	3.3	1.6	19.8	7	53	19	1.4	.160	1160.16	16	03
12 13 68	17	01	0320		13		5.6	246	198	723	182	5	203	164														

NOT REPRODUCIBLE

TABLE XIV

MILK RIVER PROJECT - SUMMARY OF EFFLUENT WATER QUALITY (CONTINUED)

DATE	STRM	SMPL	TIME	PH	TUR	TEM	DO	SUS SOL	VOL SUS SOL	TOT SOL	TOT VOL SOL	SET SOL	TOT CL	TOT HARD	CA HARD	CL DEM	BOD	TOT N	TOT P	OIL	INR C	ORG C	TOT COLI	FEC COLI	FEC STRP	CUM STORM VOLUME 10^3 FT^3	STRM	SMPL
					JTU	°F								MG/L									NO./100ML x 10^6					
01 17 69	20	01	2010					432	265	2575	325	4	1192	222	167	9.1	171	8.6	1.1	83.6	19	197	2	1.3	.059	618.77	20	01
01 17 69	20	02	2145					260	133	1667	187	2	823	487	140	7.6	116	6.0	1.0	37.5	10	96	2	0.7	.034	1086.09	20	02
01 17 69	20	03	2230					167	100	1269	184	3	557	166	124	7.8	97	4.3	0.9	35.3	10	72	3	0.4	.018	1733.28	20	03
01 18 69	20	04	0130					76	57	805	132	3	334	164	102	8.5	101	1.2	1.2	18.7	8	49	1	0.2	.014	3176.24	20	04
01 18 69	20	05	0230					75	51	595	120	2	241	142	101	9.3	91	2.3	1.0	13.5	7	49	2	0.3	.076	3239.95	20	05
01 18 69	20	06	1135					18	11	542	131	1	185	174	114	9.4	57	1.7	1.0	10.2	9	36	0	0.2	.087	6272.79	20	06
01 18 69	20	07	1440					9	9	534	108	0	192	170	113	8.5	79	0.8	1.1	7.3	9	24	1	0.2	.059	6851.59	20	07
01 19 69	20	08	1800					18	8	512	134	0	174	177	127	11.4	75	2.5	1.9	5.7	12	25	1	0.4	.063	7265.50	20	08
03 24 69	21	01	1215	8.4	50		10.4	418	196	1043	287	8	262	148	160	7.2	159	9.4	1.1	56.3	21	97	2	0.7	.420	0.00	21	01
03 24 69	21	02	1230	7.5	50		8.3	354	197	904	238	9	220	140	133	7.4	140	6.5	1.8	42.7	20	101	2	0.7	.300	201.40	21	02
03 24 69	21	03	1245	7.5	46		8.4	254	136	938	249	6	237		130	6.7	105	11.7	2.2	30.2	16	91	2	0.7	.310	585.26	21	03
03 24 69	21	04	1445	7.4	65		5.3	151	57	743	134	6	220	132	140	4.8	72	6.2	2.6	20.7	16	57	2	0.6	.260	639.15	21	04
03 24 69	21	05	1500	7.4	65		4.6	225	131	646	162	2	174	112	128	4.3	69	5.6	2.7	18.1	15	58	2	0.4	.096	790.81	21	05
03 24 69	21	06	1515	7.4	63		4.5	194	100	555	134	4	156	116	264	4.7	65	7.3	2.4	20.9	14	50	2	0.4	.190	6717.95	21	06
04 02 69	22	01	0310	7.0	73		1.5	314		900	256	6	316	175	140		129	3.4	0.7	14.5	25	83				645.45	22	01
04 02 69	22	02	0325	7.1	35		7.0	153		898	362	4	333	183	170		68	4.2	0.8	17.9	36	29				701.81	22	02
04 02 69	22	03	0417	7.1	62		5.7	177		754	186	4	248	144	135		75	4.6	0.6	17.1	15	47				842.81	22	03
04 02 69	22	04	0432	7.2	63		5.7	191		627	114	2	195	122	109		84	4.8	0.7	12.0	26	44				1015.93	22	04
04 02 69	22	05	0447	7.0	15		5.3	98		536	161	1	202	122	110		45	3.2	0.5	5.1	19	22				1254.35	22	05
04 02 69	22	06	0800	7.1			1.5	133		488	290	2	142	130	121		38	5.8	0.4	4.5	28	20				1551.72	22	06
04 17 69	23	01	1348					594	252	1023	278	9	124	158	111	8.9	103	6.4	1.2	30.2	15	117	1	1.0	.420	39.93	23	01
04 17 69	23	02	2010					360	120	680	165	2	50	114	101	6.6	42	7.7	0.8	14.1	19	71	2	0.6	.230	579.95	23	02
04 17 69	23	03	2103					44	37	423	221	0	57	218	156	9.7	14	3.3	1.0	0.0	16	24	1	0.3	.095	12134.89	23	03
04 21 69	24	01	1455	7.5	45		4.0	75	66	711	228	4	135	319	230	8.7	39	5.8	2.4	17.6	32	45	7	3.0	.260	995.80	24	01
05 18 69	25	01	0747	7.5	30		3.5	60	38	427	144	5	71	128	93	4.4	47	3.7	1.0	12.5	8	44	4	2.3	.220	569.06	25	01
05 18 69	25	02	0916	7.5	26		4.2	34	26	271	85	2	50	86	73	6.4	28	3.4	0.8	5.5	2	31	3	1.0	.091	659.30	25	02
05 18 69	25	03	1043	7.3	28		4.7	10	8	224	64	0	39	79	71	8.4	34	2.5	0.8	6.8	2	12	1	0.6	.060	1122.30	25	03
05 18 69	25	04	1212	7.2	28		4.7	30	22	217	67	2	36	76	63	8.4	39	2.6	1.0	4.6	2	25	0	0.2	.049	1956.40	25	04
05 18 69	25	05	1636	6.4	12		7.7	45	28	570	117		64	310	224	5.3	56	7.7	0.8	5.7	6	33	1	0.9	.056	6152.47	25	05
06 05 69	27	01	0340	7.4			7.0	167	95	506	328	4	82	152	103	6.1	79	7.6	1.4	12.0	18	56	6	2.9	.600	467.37	27	01
06 05 69	27	02	0820	7.4			6.4	36	23	318	195	2	71	127	91	6.0	37	5.8	1.4	9.3	12	24	1	0.7	.160	871.09	27	02
06 08 69	28	01	1037	7.5	42	36	3.5	175	76	385	271	4	63	139	117	8.2	70	7.3	0.9	15.5	14	65	5	2.9	.094	361.28	28	01
06 08 69	28	02	1448	7.5	30	36	4.3	100	47	287	186	2	49	109	95	5.0	32	3.0	0.8	17.0	2	37	2	8.3	.200	1134.55	28	02
06 08 69	28	03	1730	7.8	32	37	3.7	88	37	259	184	2	46	123	112	5.7	31	3.9	0.8	11.7	4	35	4	2.5	.100	2149.13	28	03

TABLE XIV

MILK RIVER PROJECT - SUMMARY OF EFFLUENT WATER QUALITY (CONTINUED)

DATE	STRM	SAPL	TIME	PH	TUR	TEM	DO	SUS SOL	VOL SUS SOL	TOT SOL	TOT VOL SOL	SET SOL	TOT CL	TOT HARD	CA HARD	CL DEM	BOD	TOT N	TOT P	OIL	INR C	ORG C	TOT COLI	FEC COLI	FEC STRP	CUM STORM VOLUME 10^3 FT ^{3}	STRM	SAPL
					JTU	°F	MG/L																NO./100ML X 10^6					
06 20 69	29	01	0556	7.3	52		7.2	335	115	454	185	6	72	104	97	5.9	28	4.8	0.6	8.9	12	52	2	0.5	.120	637.56	29	01
06 20 69	29	02	0752	7.4	35		8.5	89	33	233	93	0	55	68	78	6.8	6	2.6	0.7	6.1	4	21	1	0.4	.002	679.67	29	02
06 20 69	29	03	1349		24			27	30	205	77	0	55	100	95	5.8	21	4.2	0.9	5.5			1	0.3	.016	1600.37	29	03
06 23 69	30	01	1214	7.4	58		6.9	127	51	275	101	2	39	113	93	3.8	44	8.6	1.9	4.9	10	30	3	1.7	.022	466.95	30	01
07 11 69	32	01	1810	7.2	66		5.0	290	128	558	216	6	60	131	106	6.6	72		1.1	15.0	6	92	5	2.5	.230	940.31	32	01
07 17 69	33	01	1210				4.8	317	83	435	133	4	21	75	69	7.2	37	7.0	0.6	7.9	8	56	8	4.6	.360	1423.24	33	01
07 17 69	33	02	1515				4.8	158	46	381	134	2	18	102	92	7.2	43	5.9	1.1	9.5			3	2.1	.170	4071.61	33	02

One of the most significant characteristics of the overflow at the MRPS was the generally poor quality of initial storm samples. This is shown in terms of BOD and suspended solids during Event 20 in Figures 19 and 20. The plot which is fairly typical portrays the change of sewage quality with time during a relatively continuous pumping event. Note also, that chlorine demand remained relatively constant during the entire storm period, even when BOD values were extremely high.

Summaries of values of suspended solids, BOD, and chlorine demand for initial and final influent samples for all events in which two or more samples were available are summarized in Table XV. The averages of these initial and final values show the same characteristics as the continuous samples shown above. BOD and suspended solids values diminish significantly while chlorine demand remains relatively constant.

BIOLOGICAL QUALITY OF THE MILK RIVER AND LAKE SAINT CLAIR

The main objective of this work was to establish the biological character of the Milk River channel and the immediate receiving area of Lake Saint Clair. This was accomplished through ecological surveys of bottom dwelling (benthic) organisms and evaluation of the overall water quality and physical features of the area. These studies were conducted primarily to determine the condition of the Milk River Channel and Lake Saint Clair before and after full-scale treatment of combined sewer overflows with polymeric flocculants. Results of the work are presented at this time because they are helpful in gaining an understanding of the total Milk River combined sewer overflow system.

Benthic Organisms as Pollution Indicators

Aquatic organisms differ in their response to changes in water quality. Some forms survive and flourish in heavily polluted waters while other forms are unable to withstand even the slightest amount of pollution. This is true for plankton and bottom organisms as well as for fish. Benthic organisms are relatively stationary in the aquatic ecosystem and have been used extensively as indicators of water quality. Unlike fish and planktonic organisms which can selectively move throughout the aquatic environment, bottom-dwelling organisms have limited mobility and thus are valuable indicators of past and present water conditions.

A general guideline in pollution biology is that a clean water environment will support many different kinds of organisms but, the numbers of individuals representing each kind are low because of predation and competition for food and space.

Figure 19

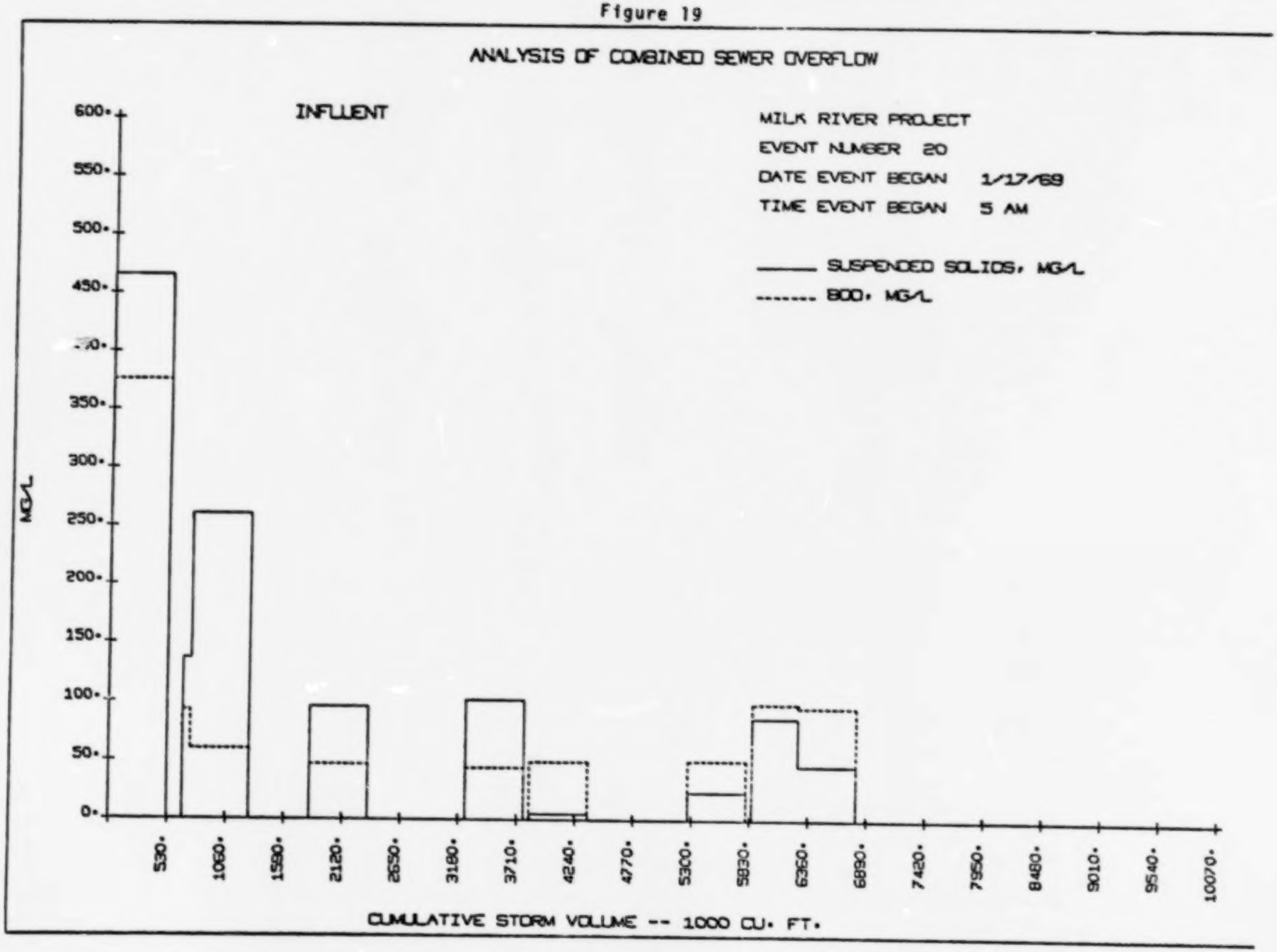

ANALYSIS OF COMBINED SEWER OVERFLOW

Figure 20

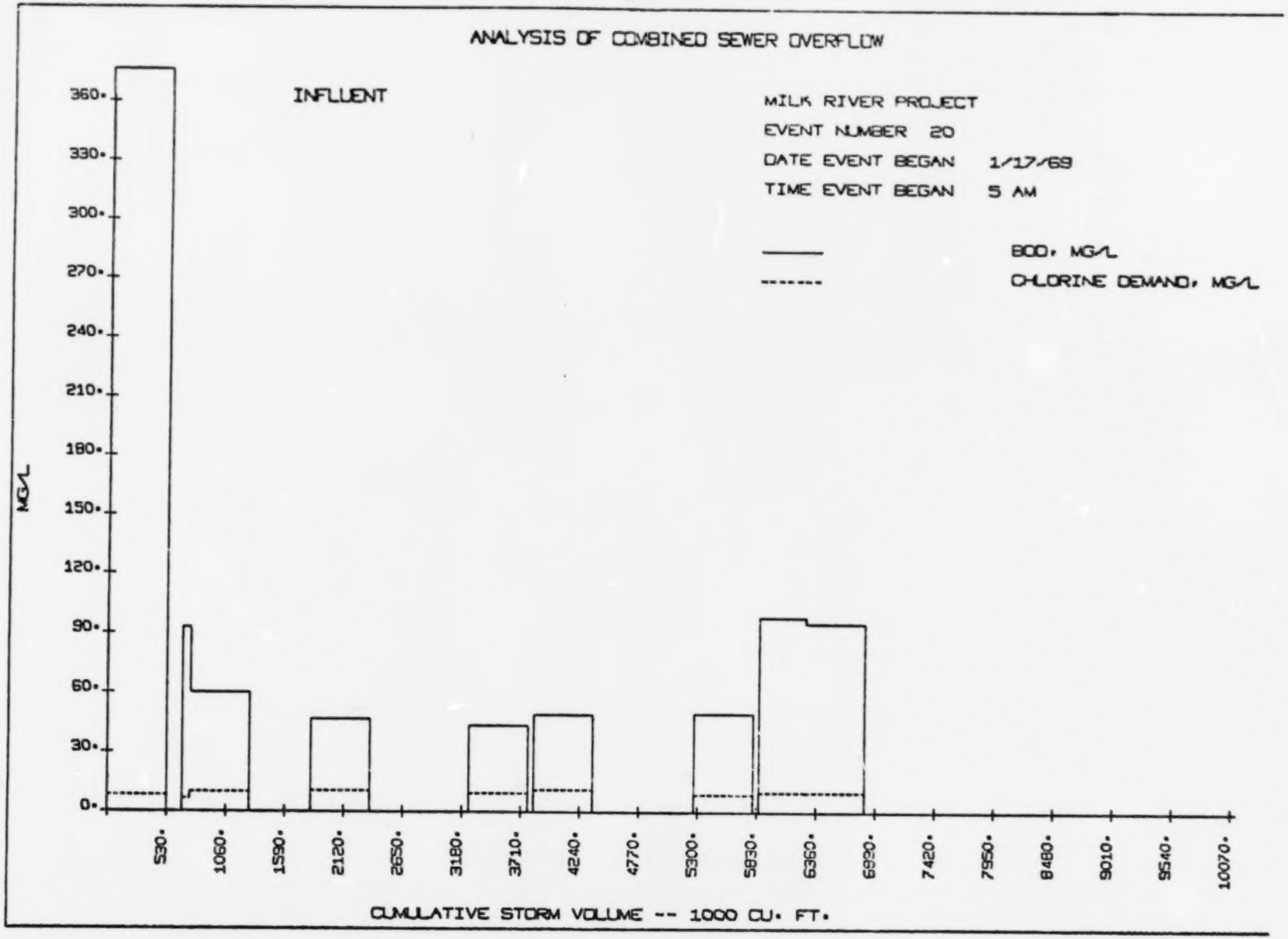

ANALYSIS OF COMBINED SEWER OVERFLOW
INFLUENT
MILK RIVER PROJECT
EVENT NUMBER 20
DATE EVENT BEGAN 1/17/69
TIME EVENT BEGAN 5 AM
BOD, MG/L
CHLORINE DEMAND, MG/L
MG/L
360.
330.
300.
270.
240.
210.
180.
150.
120.
90.
60.
30.
0.
530.
1060.
1590.
2120.
2650.
3180.
3710.
4240.
4770.
5300.
5830.
6360.
6890.
7420.
7950.
8480.
9010.
9540.
10070.
CUMULATIVE STORM VOLUME -- 1000 CU. FT.

TABLE XV

SUSPENDED SOLIDS, BIOCHEMICAL OXYGEN DEMAND, AND CHLORINE DEMAND FOR SELECTED INFLUENT SAMPLES

Event	Date	Suspended Solids mg/l Initial	Suspended Solids mg/l Final	Biochemical Oxygen Demand mg/l Initial	Biochemical Oxygen Demand mg/l Final	Chlorine Demand mg/l Initial	Chlorine Demand mg/l Final
8	7-16-68	351	156	53	40	9.2	8.0
9	7-22-68	248	110	126	24	7.6	9.6
11	8-16-68	404	46	62	32	8.2	6.1
12	8-19-68	181	110	58	31	15.1	17.6
14	10-18-68	72	26	95	31	6.8	6.6
15	11-15-68	444	57	216	120	10.9	9.3
16	11-16-68	32	50	23	33	8.9	12.2
17	12-13-68	766	116	360	52	10.2	5.3
20	1-17-69	466	46	376	96	3.5	10.1
21	3-24-69	138	24	101	35	7.7	4.5
23	4-17-69	258	72	128	17	9.0	8.9
24	4-21-69	57	52	53	38	7.3	6.4
25	5-18-69	190	40	56	43	4.5	9.2
27	6-5-69	114	44	62	44	7.0	6.5
28	6-8-69	119	78	48	18	4.7	8.3
29	6-20-69	357	53	71	48	5.4	3.5
33	7-17-69	233	169	45	40	7.2	7.6
Average		249	67	110	42	8.3	8.0

The diversity of forms is substantially reduced in environments polluted by organic wastes due to unfavorable habitat; absolute numbers of pollution tolerant organisms increase.

The life cycles of bottom-dwelling organisms are highly variable from periods of a few days to a year or longer. A qualitative examination over a long term of the different kinds of organisms present ideally would include sampling at least once during each of the annual seasons. Three ecological surveys of bottom fauna were made during the course of the project. Two surveys were made in 1968 prior to the application of flocculants for treatment. The third survey was made on August 12, 1969, after the flocculant treatment program had begun.

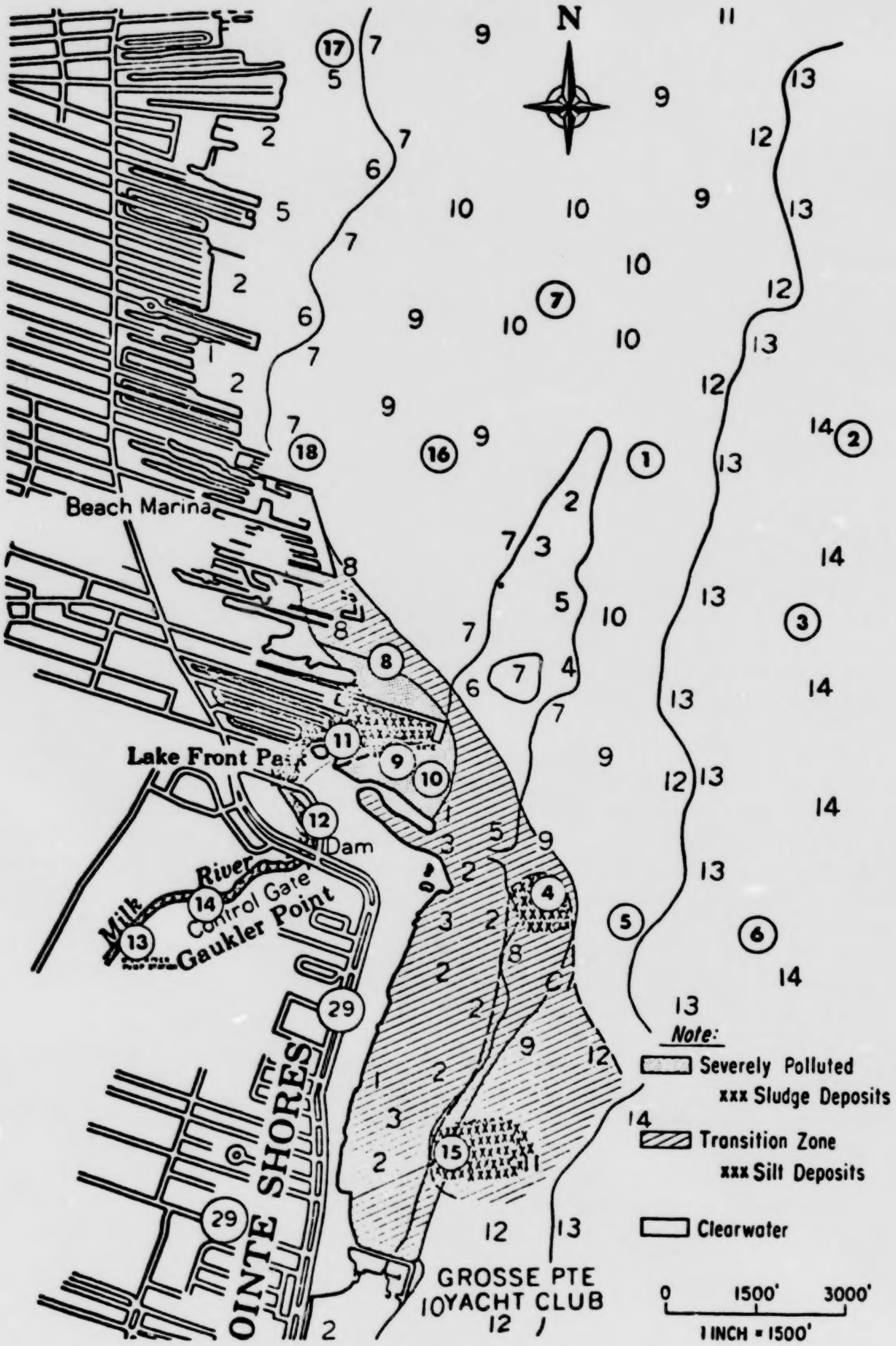
Lake and Channel Sampling Program

The biological evaluation of water quality involves comparison of data from clean water areas with those of suspected polluted areas. Sampling points were selected at significant points throughout the channel and lake area receiving the discharge of the channel (Figure 21). Every attempt was made to obtain representative samples from each area which would indicate the extremes as well as the overall condition.

Off-shore sampling points in the lake were located by two intersecting transit readings from reference positions on shore. The lake sampling points close to shore and the points in the channel were located by visual notation of physical reference points. The sampling sites therefore are relatively the same in the different surveys. Three additional samples were added to the second survey on the basis of the first results; a fourth new station was included in the last survey.

Three different samplers (Petersen, Ponar, and Ekman) were used in the surveys depending on the bottom conditions. The Ekman dredge, which is only applicable in soft silty material, was used for sampling in the channel areas. All the samples in the lake were taken using either the Petersen or Ponar dredge. Compositing was usually done to obtain a representative sampling of the biological life present. In many cases a single complete sample could not be obtained because of the unfavorable bottom conditions such as cobble and rock. All samples were carefully washed through a standard 30 mesh screen and preserved with formalin in labeled jars for later laboratory examination. Observations of the nature of the bottom condition at each sampling point were recorded and water samples were collected. Dissolved oxygen and bacteriological determinations were made at each sampling point.

FIGURE 21
MILK RIVER CHANNEL AND ESTUARY WITH ZONES OF POLLUTION


N
Beach Marina
Lake Front Park
Dam
Milk River
Control Gate
Gaukler Point
POINTE SHORES
GROSSE PTE
YACHT CLUB
Note:
Severely Polluted
xxx Sludge Deposits
Transition Zone
xxx Silt Deposits
Clearwater
0
1500'
3000'
1 INCH = 1500'

Additional qualitative sampling of the bottom was done to determine the degree and extent of bottom deposition of organic material. Qualitative sampling in the lake consisted of visual observation of numerous dredge samples in selected areas. The qualitative sampling in the channel bottom was more extensive and included the development of channel profiles at various cross sections. These profiles were developed from data obtained with a long indexed pole at spaced intervals on selected cross sections of the channel.

Benthic Character of Milk River and Lake Saint Clair

A composite summary of the ecological surveys is presented in Table XVI. A summary of the dissolved oxygen, coliform, and bottom material descriptive data are contained in Table XVII.

Based on these findings, each sampling station was subjectively rated to be in a clean water condition, a polluted condition, or in an intermediary transition condition.

There were diverse populations, including a variety of clean water organisms, at Stations 1, 2, 3, 5, 6, 7, 16, 17 and 18. These locations were designated as clean water stations. The life-forms at Stations 1 and 7 seem to be depressed slightly. This is likely a result of the nature of the substrate rather than of water quality. The bottom at these stations was gravel and rock. This type of bottom supports a specific type of life and collection of representative samples is difficult. These off-shore samples represent the natural benthic populations in Lake Saint Clair which are relatively influenced by pollution.

Stations 4 and 15 fall into the zone of transition. Although significant numbers of life-forms are present, they are being suppressed by some form of pollution. The number of clean water organisms present at these stations is reduced to only a few; pollution-tolerant forms have increased greatly in number. The bottom material at Stations 4 and 15 consisted of a deposition of fine organic ooze in contrast to the sandy clay of the clean water points. The pollution in this area is more probably in the form of substrate alteration rather than deterioration of water quality.

The remaining sampling points (Stations 8 through 14) show definite signs of gross organic pollution. The diversity of species is reduced to only a few tolerant organisms which are evident in great numbers. These stations located in the Milk River channel and the immediate areas of the lake are definitely greatly influenced by overflow from the MRPS. The dissolved oxygen is low and very high bacteria counts

TABLE XVI
COMPOSITE SUMMARY OF ECOLOGICAL SURVEYS

Station	Ave. No. Organisms	Ave. Organism Diversity	Subjective Classification
1	30	8	C
2	91	12	C
3	92	14	C
5	64	9	C
6	109	15	C
7	34	8	C
16	128	12	C
17	124	10	C
18	-	12	C
4	199	8	T
15	124	8	T
8	117	5	Po
9	75	3	Po
10	59	2	Po
11	227	1	Po
12	116	1	Po
13	61	1	Po
14	64	1	Po

C = Clean water
T = Transition
Po = Polluted

TABLE XVII

QUALITY AND CHARACTER OF BIOLOGICAL SAMPLING LOCATIONS

Sample Station	SPRING SURVEY - 1968			FALL SURVEY - 1968			QUALITATIVE SURVEY - 1969		
	Dissolved Oxygen mg/l	Total Coliform #/100 ml	Observation of Bottom Material	Dissolved Oxygen mg/l	Total Coliform #/100 ml	Observation of Bottom Material	Dissolved Oxygen mg/l	Total Coliform #/100 ml	Observation of Bottom Material
1	13.1	<100	Clean gravel and rock	8.9	125	Gravel and rock	-	-	-
2	12.8	<100	Clay and sand	9.2	102	Clay and sand	-	170	Clay and sand
3	12.0	<100	Clay and sand	9.0	112	Clay and sand	-	-	-
4	12.6	400	Gravel, sand, slight silt	9.3	78	Heavy organic silt ooze	8.2	-	Heavy organic silt ooze
5	12.6	<100	Gravel, sand and clay	9.4	65	Clay and sand	-	-	-
6	13.6	<100	Clay and sand	9.1	30	Clay and sand	8.2	60	Clay and sand
7	13.6	<100	Gravel and sand	9.3	134	Clean sand	-	-	-
8	10.2	870,000	Debris, some sewage odor	8.7	100	Organic ooze, sewage odor	-	170	Organic ooze, sewage odor
9	10.0	76,000	Clean sand	9.1	34	Sand, some silt	7.6	390	Dark sand slight odor
10			Gravel and sand	9.3	94	Sand, some silt	-	-	-
11	8.6	57,000	Sludge ooze, sewage odor	8.2	150	Sludge ooze, sewage odor	-	-	-
12	4.8	13,000	Sludge ooze, sewage odor	6.8	10,400	Sludge ooze, sewage odor	6.2	3700	Sludge ooze, clay, debris
13	4.2	11,000	Sludge ooze, sewage odor	5.7	1,800	Sludge ooze, sewage odor	-	-	-
14	-	-	Sludge ooze, sewage odor	5.8	2,300	Sludge ooze, sewage odor	-	-	-
15	-	-		9.2	TN	Organic silt, heavy ooze	7.2	-	Heavy organic silt ooze
16	-	-	-	9.4	128	Gravel and sand	-	54	Gravel and sand
17	-	-	-	9.7	N.C.	Rock and sand	-	110	Gravel and sand
18	-	-	-	-	-	-	-	270	Debris and sand

are present at these stations. The partially exposed bottom of the channel approximately midway between the Pumping Station and Lake Saint Clair is shown in Figure 22. This condition existed during a partial dewatering of the channel in the summer of 1968.

Figure 22

Bottom of Milk River Channel Partially Exposed During Dewatering

Slight improvements in the conditions of the Milk River channel were evident in results from the fall surveys. A few tolerant midge larvae in addition to the dominant population of sludge worms were evident. This is probably a direct result of physical recruitment from channel flushing during the summer months. Life in the channel is periodically replenished by new organisms from the cleaner lake waters during the flushing process.

The quality of the lake water is undoubtedly influenced by other effluents including effluent from the Chapaton Combined Sewer overflow which enters the lake just north of the Milk River channel. Sample Station 8 is located directly off the mouth of this drain. Results for samples from this station vary considerably between surveys. This station was initially characterized by a few cleanwater organisms and moderate diversity in the population placing it in no worse than a transition condition. Greatly reduced diversity and definite indications of severe pollution were evident in the fall surveys. The lake area near the Chapaton Drain prior to the spring survey was disrupted due to the construction of a new pumping station at that time.

The influence of the poor quality water from Milk River channel on the biota of the lake is shown by looking at Stations 11, 10, 4, 5, and 6. Station 11 which is located directly in the mouth of the channel is grossly polluted with heavy organic sludge deposits and is characterized by great numbers of tolerant sludge worms. Moving out into the bay to Station 10 beyond the outfall pipe, the sludge deposits are gone, undoubtedly due to the wind and wave action, and yet the water quality is such that poor biological populations still exist. Further out into the lake, at Station 4, a transition to better conditions is definitely evident. A few cleanwater organisms are present and the overall population has become more diverse.

At Stations 5 and 6 cleanwater organisms are abundant and the population has become more diverse. Station 6 is even better than 5 but these differences could likely be in sampling variation.

The conditions found at each station and the overall interpretations are shown in Figure 21. The limits of these areas, particularly in the transition zone, cannot be sharply defined without further work. Further discussion of the bottom depositions are made in the next section.

Qualitative Observations

A qualitative survey of the nature of the bottom material in the channel and lake was made by observing conditions at the sampling stations and at other strategic points. As already noted, the bottom of the Milk River channel is extensively covered with sludge deposits. The same conditions were observed in qualitative samples taken from other boat canals which enter directly into the bay area near the discharge of the Milk River. Heavy sludge deposits were also found in the bay throughout the protected area created by the pier shown in Figure 21. No sludge deposits were observed south of a line extending from the end of the pier to a point south of sampling point 11 at the south side of the channel mouth. The wind and wave action apparent washes clean the area south of this line. Deposition is pushed back into the channels or out into the lake depending upon wind direction.

Channel profiles developed from cross-sectional depth sampling are shown in Figure 23. Sludge deposits exist at any location where the flow conditions in the channel are favorable for deposition of material, e.g. the inside bank of curves, back eddies, etc. The extent of sludge deposition was so significant that it was impossible to assess changes brought about by chemical treatment of the storm overflow in the Milk River.

Effect of Milk River Channel on Storm Overflows

The Milk River Channel has a unique role in the overall effect on the combined sewer overflow from the MRPS. As shown, significant sludge deposits are in evidence in the channel. These deposits undergo rapid decomposition during the summer months causing serious odor problems and depletion of dissolved oxygen in the entire length of the channel.

In an effort to evaluate the cause of these deposits in some quantitative way, the flow conditions in the channel were analyzed considering the conditions of pumping which prevailed from 1960 through 1969. Before summarizing the results of the analyses, however, it is necessary to point out certain physical characteristics of the channel.

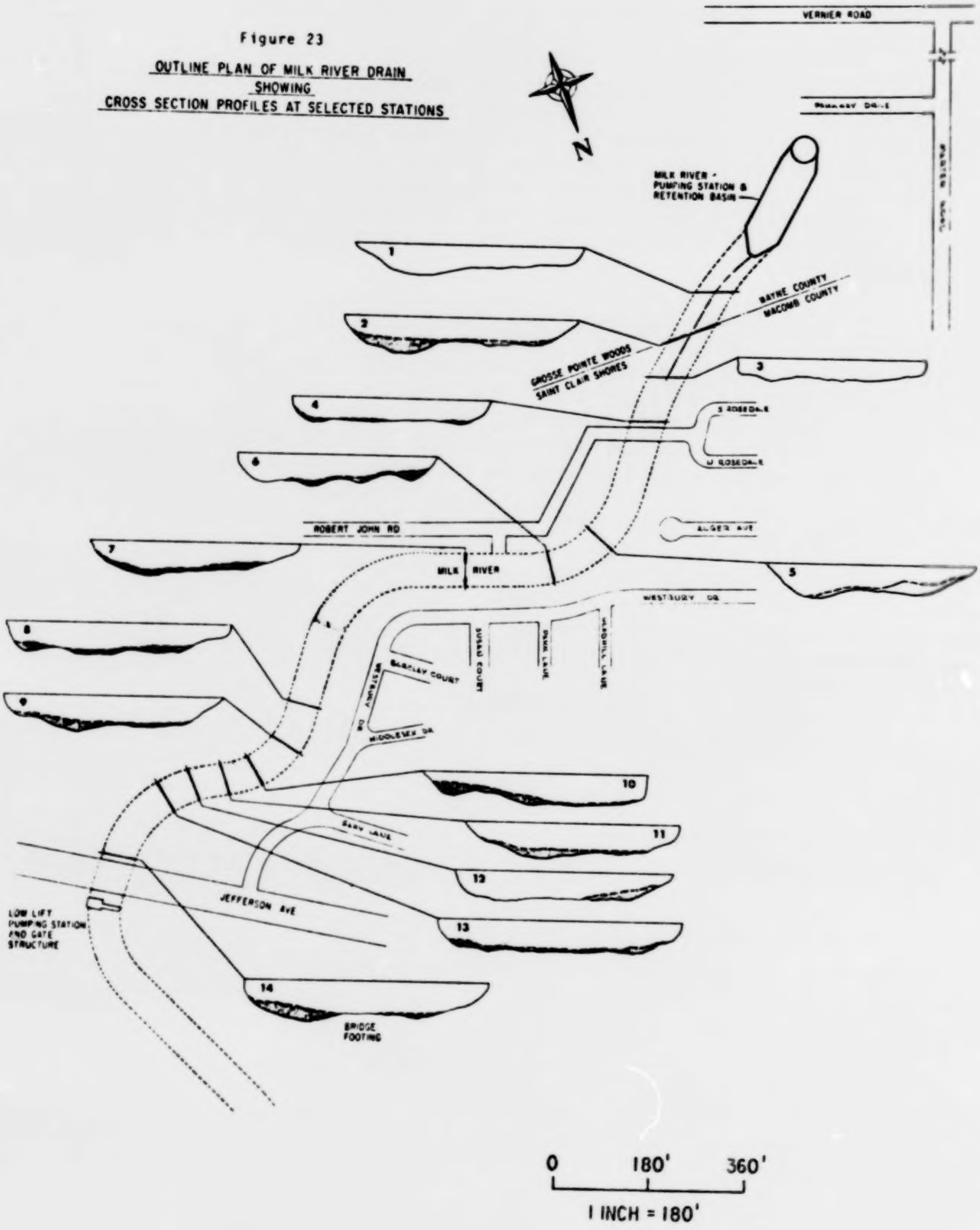
TABLE XVIII

PHYSICAL CHARACTERISTICS OF THE MILK RIVER CHANNEL

Length (to Lake Saint Clair)	6,000 ft
Area (cross-sectional)	625 sq ft
Estimated Volume (to Lake Saint Clair)	3,750,000 cu ft 28,100,000 gal
Mean Depth	7.5 ft
Volume of Milk River Retention Basin	512,000 cu ft 3,830,000 gal

Figure 23

OUTLINE PLAN OF MILK RIVER DRAIN
SHOWING
CROSS SECTION PROFILES AT SELECTED STATIONS


VERNIER ROAD
N
MILK RIVER - PUMPING STATION & RETENTION BASIN
WAYNE COUNTY
MACOMB COUNTY
GROSSE POINTE WOODS
SAINT CLAIR SHORES
S ROSEDALE
N ROSEDALE
ROBERT JOHN RD
ALGER AVE
MILK RIVER
WESTBURY DR
SUSAN COURT
PARK LANE
BARCLAY COURT
WESTBURY DR
MIDDLESEX DR
JEFFERSON AVE
LOW LIFT PUMPING STATION AND GATE STRUCTURE
BRIDGE FOOTING
1
2
3
4
5
6
7
8
9
10
11
12
13
14
0
180'
360'
1 INCH = 180'

DATE: OCTOBER, 1968

Based on these approximations and an analysis of the frequency distribution of rate of pumping and pumping volumes, the following data were calculated.

TABLE XIX
RATE OF PUMPING

Flow (cfs)	Rate Occurrence (%)	Channel Velocity (fps)	Detention (min)	
			Channel	Basin
187*	50	0.30	335	46
305	61	0.49	205	28
410	91	0.66	152	21
610	93	0.98	102	14
715	95	1.14	88	12
2450	100	3.92	25	4

*Geometric Mean

TABLE XX
VOLUME OF PUMPING

Total Volume Pumped		Volume Occurrence (%)
Cu Ft	Gal	
1,000,000	7,480,000	40.3
1,200,000*	8,970,000	50.0
2,000,000	14,960,000	61.3
3,000,000	22,440,000	75.2
4,000,000	29,920,000	83.4

*Geometric Mean

The Milk River Channel is serving as an effective treatment system in its own right as clearly shown by the above data and supported by the results of the channel sampling program. It is probably significantly more effective in preventing fresh solids from getting into Lake Saint Clair than the existing retention basin. Of the combined sewer overflows which occurred from 1960 through 1969, 95 percent produced channel velocities equal to or less than 1.14 fps while 91 percent of the overflows produced velocities equal to or less than .66 fps. These velocities alone would permit deposition of most sewage type solids, but another important factor is noted in Table XX. In over 83.4 percent of the storms, the volume of the storm overflow was not sufficient both to fill the retention basin and to displace the volume in the Milk River Channel up to Lake Saint Clair. Since there is no dry weather flow in the Milk River, most suspended solids pumped into the channel are allowed to settle under quiescent conditions for periods of from several hours to several weeks.

SECTION 6

TREATMENT OF COMBINED SEWAGE WITH POLYMERIC FLOCCULANTS

One of the primary objectives of the Milk River Contract was to study the problem of chemically treating large volumes of continuously flowing combined sewage at the Milk River Pumping Station (MRPS) with relatively low concentrations of organic water-soluble polymeric flocculants and measure the clarification potential of the process. The work involved a program of laboratory, pilot, and full-scale testing designed to demonstrate the practicability of the process and to outline the problems associated with the evaluation and operation of such a system.

In general, this type of chemical treatment can be divided into two operations, flocculation and sedimentation. Flocculation involves mixing the combined sewage with a polymeric flocculant under proper conditions to generate a floc with desirable settling properties[1]. Sedimentation involves the removal of the flocculated material along with other heavy solids under quiescent conditions. The clarified overhead is discharged to the receiving water. Flocculation studies at the MRPS included preliminary screening of flocculants in jar tests, assessment of optimum flocculant systems in a long-tube sedimentation column, and full-scale application of flocculants. The sedimentation studies involved measurement of settling rates of samples of storm flow flocculated using a long-tube settling column and evaluation of retention efficiencies of the basin using a hydraulic model of the Milk River Retention Basin.

FLOCCULATION STUDIES

A study of flocculation of any waste with chemical flocculants requires evaluation of at least three factors:

a. The capability of the material(s) selected to react with the wastewater and form a settleable floc.

b. The reaction times required between chemical addition, dispersion, and floc formation.

c. The conditions of mixing required to develop satisfactory or optimum floc.

Experience in the treatment of sanitary sewage, rivers, and industrial wastes has demonstrated the importance of providing adequate time for dispersion of flocculants. The time required for dilute but viscous polymeric flocculant solutions to disperse and react in most wastewaters under the condition of flash mix (high shear gradient) is approximately 0.5 to 1.0 minutes. Flocculation of most wastewaters can be accomplished under conditions of gentle mixing in an effective time of from 1 to 3 minutes. It has also been found that a reasonable estimate of the potential clarification efficiency of a polymeric flocculant treatment system can be obtained by measuring overhead clarity in a sample flocculated and settled in a 1500 ml beaker under standardized conditions of mixing.

The flash mix required to accomplish polymeric flocculant dispersion corresponds to an approximate shear gradient greater than 50 sec^{-1} or a Phipps and Bird lab stirrer speed of approximately 90 rpm in a 1500 ml beaker. A shear gradient of approximately 10 sec^{-1} or a Phipps and Bird mixing stirrer speed of about 40 rpm in a 1500 ml beaker is required for flocculation. The effective retention times and mixing requirements for most polymeric flocculants in wastewater systems are summarized in Table XXI.

TABLE XXI

CONDITIONS FOR DISPERSION AND FLOCCULATION

Criterion	Dispersion	Flocculation
Minimum time	0.5	1
Optimum time	1	3
Mixing conditions	Flash	Gentle
Approximate shear gradient (sec^{-1})	50-80	>10
Gang stirrer setting (rpm)	90	40

The initial work at Milk River was devoted to screening various types of polymeric flocculants for their ability to react with combined sewage to form a settleable floc using these guidelines for mixing conditions and reaction times.

JAR TEST SCREENING

The standard conditions for jar testing adopted for the screening of flocculant efficiencies were: one minute of flash mix at 100 rpm, three minutes of flocculation mix at 40 rpm, and five minutes of settling at 10 rpm using a Phipps and Bird stirrer. These values are approximately the same as the effective values shown for optimum conditions in Table XXI. A 1000 ml volume of the material to be flocculated was contained in each of six 1500 ml beakers. Qualitative measurements of flocculation rate, floc size, settling rate, and overhead clarity were routinely recorded. Initial sample turbidity and final overhead turbidity were measured instrumentally.

The screening studies were designed to evaluate the potential effectiveness of polymeric flocculants having a broad range of chemical characteristics. The PURIFLOC products investigated during this study are classified according to ionic properties and basic formulation in Table XXII.

TABLE XXII

CLASSIFICATION OF PURIFLOC FLOCCULANTS

	Anionic		Cationic		Nonionic
A21	Sodium Polystyrene Sulfonate	C31	Polyamine	N11	Polyacrylamide
A22	Polyacrylamide	C32	Polyethylenimine	N12	Polyacrylamide
A23	Polyacrylamide	ET 721	Polyacrylamide*		

*Experimental flocculant, no PURIFLOC designation

The eight flocculants were evaluated at various concentrations for potential clarification efficiency utilizing the jar testing procedures outlined above. The tests were conducted on samples obtained at various periods during Events Nos. 1-8.

The results of the laboratory jar tests in terms of flocculation efficiency as measured by overhead turbidity in treated and untreated samples are summarized in Table XXIII. The effect of storm period on relative influent turbidity and flocculation efficiency are noted. The following conclusions were made based on these initial screening tests:

1. The anionic and nonionic polymeric flocculants exhibited no significant flocculation activity and were not effective in reducing influent turbidity.
2. Two of the cationic flocculants, PURIFLOC C31 and PURIFLOC C32, consistently exhibited excellent flocculation activity and effective turbidity removals at concentrations of approximately 10 mg/l during the early to mid-storm periods when suspended solids loadings were relatively high.

TABLE XXIII

RELATIVE FLOCCULANT ACTIVITIES AS MEASURED BY TURBIDITY

Event No.	Raw	Settled Control	A21*	A22*	A23*	C31*	C32*	N11*	N12*	ET-721	BAROID HECTORITE + C32*
			(2.0 mg/1)			(15 mg/1)		(2.0 mg/1)		(10 mg/1)	15 + 10 mg/1 res.
1	375	350	350	-	280	62	67	350	350	-	-
1	450	425	425	-	425	95	100	-	-	375	-
1	160	110	110	-	110	110	110	-	-	-	-
2	235	200	-	-	-	75	90	-	-	175	-
2	160	100	100	100	100	85	76	100	100	-	-
3	260	245	220	220	230	30	62	-	245	-	-
3	280	203	-	-	-	65	48	-	203	-	40
3	215	160	-	160	160	90	60	160	-	100	40
4	170	160	160	-	145	60	55	-	-	-	22
4	98	95	95	95	95	80	80	95	95	90	35
5	235	220	-	-	210	68	55	-	215	-	45
6	178	125	120	-	-	72	68	125	-	-	21
7	258	220	-	-	-	50	40	-	-	-	-
7	220	190	190	-	-	90	52	-	170	135	-
7	290	265	260	260	260	70	50	180	240	140	10
7	200	195	190	-	190	98	70	170	190	120	17
7	175	125	-	-	-	70	60	-	-	-	35
8	370	180	155	155	-	62	90	130	130	60	47
23	100	76	76	-	76	75	75	-	76	-	65
25	180	155	155	155	-	85	80	140	155	-	50

*PURIFLOC products

3. During the mid- to late-storm periods, when suspended solids loadings were relatively low, PURIFLOC C31 and PURIFLOC C32 exhibited relatively poor flocculation activity.

4. Final overhead turbidity was relatively constant after treatment by PURIFLOC C31 and PURIFLOC C32 during all storm periods; final overhead quality is apparently independent of initial sample turbidity.

The characteristic decrease in suspended solids and turbidity of combined sewage during the course of a storm overflow limited the potential effectiveness of a single flocculant treatment system. Previous experience had shown that in systems of low solids, floc building nuclei could be added to supplement the natural solids and achieve high efficiency clarification. During Events Nos. 3 and 4, two montmorillionite clays, BAROID CERCLAY® and BAROID HECTORITE®, were tested as coagulant aids. The BAROID HECTORITE clay, a calcium-magnesium silicate, was found to be far superior in effectiveness to the BAROID CERCLAY, an aluminum silicate.

The significant improvement in removal of turbidity obtained by addition of BAROID HECTORITE clay to PURIFLOC C32 over the PURIFLOC C32 system alone is shown in Figure 24. During screening studies of the clay-cationic flocculant system a minimum dosage of approximately 15 mg/l of clay was necessary to achieve a significant increase in effectiveness over the cationic flocculant alone. Clay concentration was critical especially when compared with effective polymeric flocculant concentrations. The optimum concentration of polymer flocculant dosage is quite broad, even in the dual systems. It was also observed that clay addition was more effective when it preceded flocculant addition by approximately 30 seconds of intervening mixing.

In addition to the clay-cationic polymer flocculant systems, several other dual and tertiary systems were also evaluated as well as several cationic blends which exhibit the properties of a dual cationic-anionic system. Those systems which provided flocculation activity and turbidity removal efficiencies greater than that of the cationic polymer flocculants alone were:

a. Ferric chloride + PURIFLOC A23

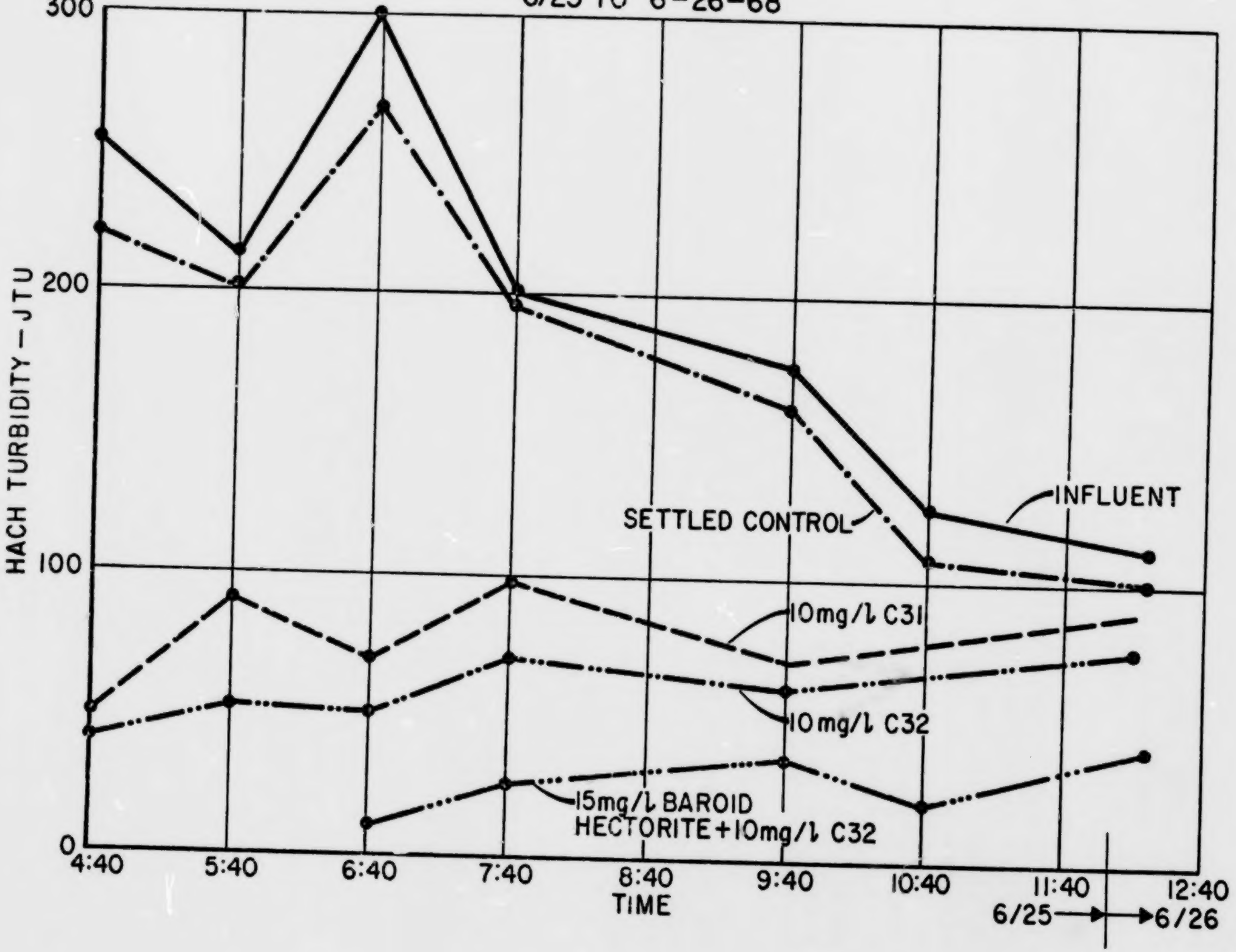
b. PURIFLOC C31 + PURIFLOC A23

c. BAROID HECTORITE clay + PURIFLOC C31 + PURIFLOC A23

d. BAROID HECTORITE clay + PURIFLOC N11 + PURIFLOC C31

Figure 24

MILK RIVER STORM EVENT # 7
6/25 TO 6-26-68



It was also noted during the course of these system screening studies that very rapid formation of floc occurred when anionic and cationic polymeric flocculants were used in a dual system. Rapid floc formation also occurred when PURIFLOC N11 was used between the addition of clay and the addition of the cationic flocculant. This capability to significantly increase the rate of floc formation can be utilized to great advantage in systems where flocculation time is short. Increased flocculation rates appear to be accompanied by decreases in solids capture efficiency.

The following conclusions were made from the dual and tertiary system polymer screening studies:

1. Coagulant aids, specifically 15 mg/l of BAROID HECTORITE clay, can be utilized with the cationic flocculant systems to produce significantly improved turbidity removal from the MRPS combined sewage.
2. The rate of floc formation can be increased significantly by the addition of anionic polymer flocculants to the cationic flocculant systems, but may result in a corresponding decrease in solids capture efficiency.

FULL-SCALE APPLICATION

Treating a periodic waste flow of up to a million gallons per minute with polyelectrolyte flocculants is a problem of monstrous proportions. It was therefore necessary to design a flocculant feed system on a much larger scale than had been previously applied in the treatment of continually flowing municipal wastewaters. A frequency distribution was developed for the rates of pumping for eight previous years of record at the MRPS. The combined sewage flow was less than or equal to 680 cfs (305,000 gpm) for 90 percent of the time; the flow was ≤1100 cfs (493,000 gpm) for 95 percent of the time. The maximum flow possible is 2450 cfs (1,100,000 gpm). A cost-benefit analysis was made to determine the amount of capital required to treat 90, 95, and 100 percent of anticipated volume. It was decided that the upper limit of flow to be treated with flocculants would be 1100 cfs (the 95 percent occurrence). The capital outlay to treat the final 5 percent of the flow situations is double that for the first 95 percent of flow.

Two different chemical dispersal systems were required: one for dispersion of dry anionic and nonionic polymers, and the other to dilute and disperse liquid cationic polymer. A sub-contract was let to the Hague Equipment Company of Evansville, Indiana, to design and construct an automatic dispersing unit for dry chemicals capable of a continuous flow of 100 gpm. Previous experience by The Dow Chemical Company with this supplier permitted direct expansion of the basic components of this automatic disperser from the usual 25 gpm capacity up to a 100 gpm capacity. Operation of this unit for dispersing dry anionic flocculants was trouble-free.

Dilution of the relatively large amounts of cationic flocculants was best accomplished using an in-line blender. The cationic flocculant in liquid form was first pumped through a positive displacement pump into the in-line blender. Water was introduced and the two phases instantly mixed in a high shear enclosure. The polymer solution was stored after dilution in a holding tank. Depending on the polymer concentration required, flow rates up to 100 gpm were possible using the in-line blender. This system proved to be very effective for dispering cationic flocculants during several events.

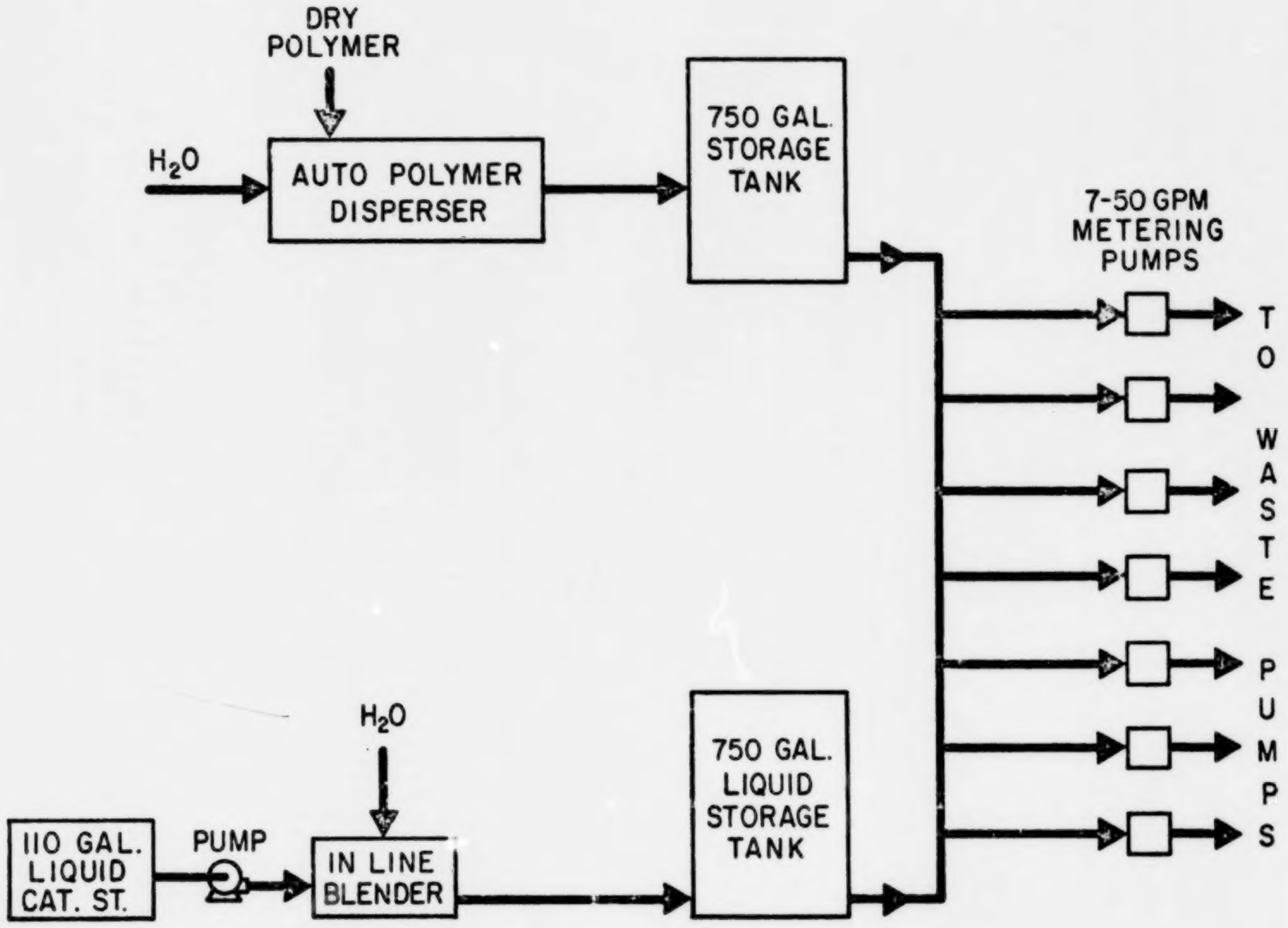
Seven variable speed chemical metering pumps were required to pump the dispersed polymer into any combination of the seven storm lift pumps. Gear pumps with a capacity of 50 gpm fitted with variable speed drives were reliable and effective. A schematic of the complete feed system is presented in Figure 25.

Based on the initial jar test evaluations, PURIFLOC C31 and PURIFLOC C32 were selected for the first full-scale feedings of flocculant. There were four events (Nos. 16, 23, 27, and 30) in which flocculants have been applied throughout. There were six other events in which continuous feeding was not achieved because of various technical problems or sampling difficulties.

Selection of the proper point for flocculant addition was critical. The difficulty of selection is compounded by the relative high flow rates and minimal detention times available for flocculation at the MRPS. Flocculant was first fed full-scale during Event 15. A dosage of 18.8 mg/1 PURIFLOC C32 was fed for the entire 21 minutes of the third pumping. The point of flocculant addition was located about half way up the casing on the discharge side of storm pump #3. Samples from various basin points during this feed contained small-medium sized flocs. Two problems were immediately apparent: (1) a significant portion of floating floc was noted, and (2) much floc was "popping" over the effluent weirs.

Figure 25

SCHEMATIC – POLYMER FEED SYSTEM


DRY POLYMER
H_2O
AUTO POLYMER DISPERSER
750 GAL. STORAGE TANK
7-50 GPM METERING PUMPS
TO WASTE PUMPS
H_2O
110 GAL. LIQUID CAT. ST.
PUMP
IN LINE BLENDER
750 GAL. LIQUID STORAGE TANK

A condition of floating floc is usually indicative of excessive concentrations of flocculant or improper mixing conditions with possible air entrainment. Results from jar tests conducted during this event demonstrated that 20 mg/l PURIFLOC C32 was not an overdosed condition. Improper mixing was concluded. Careful examination of the floating flocs showed small bubbles of air attached to each floc particle. This air was probably trapped on the flocs during the violent aeration immediately after discharge of the pumps onto the distribution plane of the basin (first concrete apron).

The addition point was changed to a position near the second apron downstream of the first apron. Visual observation of the basin operation showed that the aeration effect was reduced at this new addition point. This second addition point was used when PURIFLOC C32 was fed at a dosage of 15 mg/l during Event 17. Very small flocs were observed in the basin; good flocculation was apparent in the jar tests. The shear gradient was apparently too low at this addition point to effect proper flocculation.

The third and last addition point tested was located at the first apron in the vicinity of the violent agitation created by the pump discharge. The shear gradient was relatively high at this point. Very good rapid mix conditions were available for proper dispersal of flocculant. The area of the second apron provided a relatively lower shear gradient in which floc could grow. Excellent flocculation was noted using this addition point. The previously observed flotation problems were minimal. This addition point was utilized for all of the remaining full-scale additions.

Samples collected in the basin during all subsequent applications of flocculant contained medium to large flocs. Capture of fine solids into the larger flocs was fair. The single flocculant systems of PURIFLOC C31 or PURIFLOC C32 were used during the early periods. Visual observation of samples collected in the vicinity of the overflow weir indicated extensive carry-over of floc.

It was known that acceptable flocculation was occurring in the basin, but proper sedimentation of these flocs was not occurring. How could flocculation be quantified? A good measure of the extent of flocculation of a particular system was afforded by the measurement of the volumetric settleable solids; i.e, the volume of settleable matter contained in an Imhoff cone recorded after one hour of sedimentation expressed as volume of sludge/volume of sewage (ml/l). Untreated influent samples were initially assumed to contain less settleable solids than a treated sample from the basin or the effluent sample. Little sedimentation was thought to be occurring in the basin. This was not always the case, however, even when several samples were averaged to minimize the inherent time lag between influent and effluent samples.

Selected data from samples collected by the automatic samplers during Event 23 are shown in Table XXIV.

TABLE XXIV

SETTLEABLE SOLIDS FOR SELECTED
STORMWATER SAMPLES OF EVENT 23

	Settleable Solids, ml/l	
	Influent	Effluent
First pumpage	3.2	8.7
Second pumpage	1.7	2.5
Third pumpage	1.5	1.5

The average values of five samples collected in intervals at both the influent and effluent stations for the same Event showed an influent value of 9.7 ml/l and an effluent value of 7.9 ml/l. This positive degree of removal is not in agreement with the above data.

Another possible method to quantify the flocculation process was to measure sludge depths and develop profiles of the basin bottom. Because of dramatic quality changes both within single events and between two or more events, no true "control" situation existed, and this method was impractical.

Because of the inherent settling inefficiency of the Milk River basin and the practical difficulty of controlling flocculant concentrations during short periods of pumping, optimum full-scale flocculation was difficult to achieve. The extreme variations of sewage quality and the inherent inefficiency of the basin made performance evaluation without a control impossible. In addition, the model and the long-tube sedimentation portions of the study had shown that the hydraulic inefficiencies of the system will not permit satisfactory settling to occur.

For these reasons, considerable effort was devoted to the long-tube settling tests in order to establish those parameters necessary to engineer and design an efficient basin for chemical treatment of combined sewer overflow.

STREAMING CURRENT DOSAGE CONTROL

An automatic control device for chemical addition would greatly aid application of flocculants for two reasons: (1) a shortage of qualified operators in the waste treatment field, and (2) the extreme variation in waste quality. Previous experience with the streaming current detector within The Dow Chemical Company[1,2] has shown that it provides a means of controlling the concentration of cationic flocculants added to domestic sewage. This device develops a streaming flow of fluid confined in an annular space which shears the electrical double layer. The movement of this diffused double layer creates a measurable streaming current indicative of the character of the electrical charges of colloidal suspensions.

Streaming current measurements were made during laboratory flocculations of a sample collected during Event 27. The results are presented in Table XXV. For Influent No. 1, when the solids concentration was high, it was apparent that the streaming current closely followed the traditional charge approach to flocculation; that is, optimum flocculation was seen to occur at or near (slightly on the positive side) the point of zero streaming current. It was interesting to note the non-linearity of the streaming current with increasing dosages of PURIFLOC C31. This response is shown in Figure 26. The streaming current of raw sewage is usually in the range of -20 to -25 microamps.

For Influent No. 2, with a much lower solids loading, the condition of near zero streaming current did not relate to acceptable flocculation. It was seen that 15 mg/l PURIFLOC C31 reversed the charge of the system, but that clay was necessary to provide bridging sites in this low available surface-site system.

Optimum flocculant concentration apparently could be related to streaming current, but optimum flocculation during low solids conditions required a given quantity of flocculant aid in addition to the polyelectrolyte. Since the dosage of flocculant aid from previous jar testing appeared to be constant, an automatic system to predict polymer demand and feed polymer seems practicable.

TABLE XXV

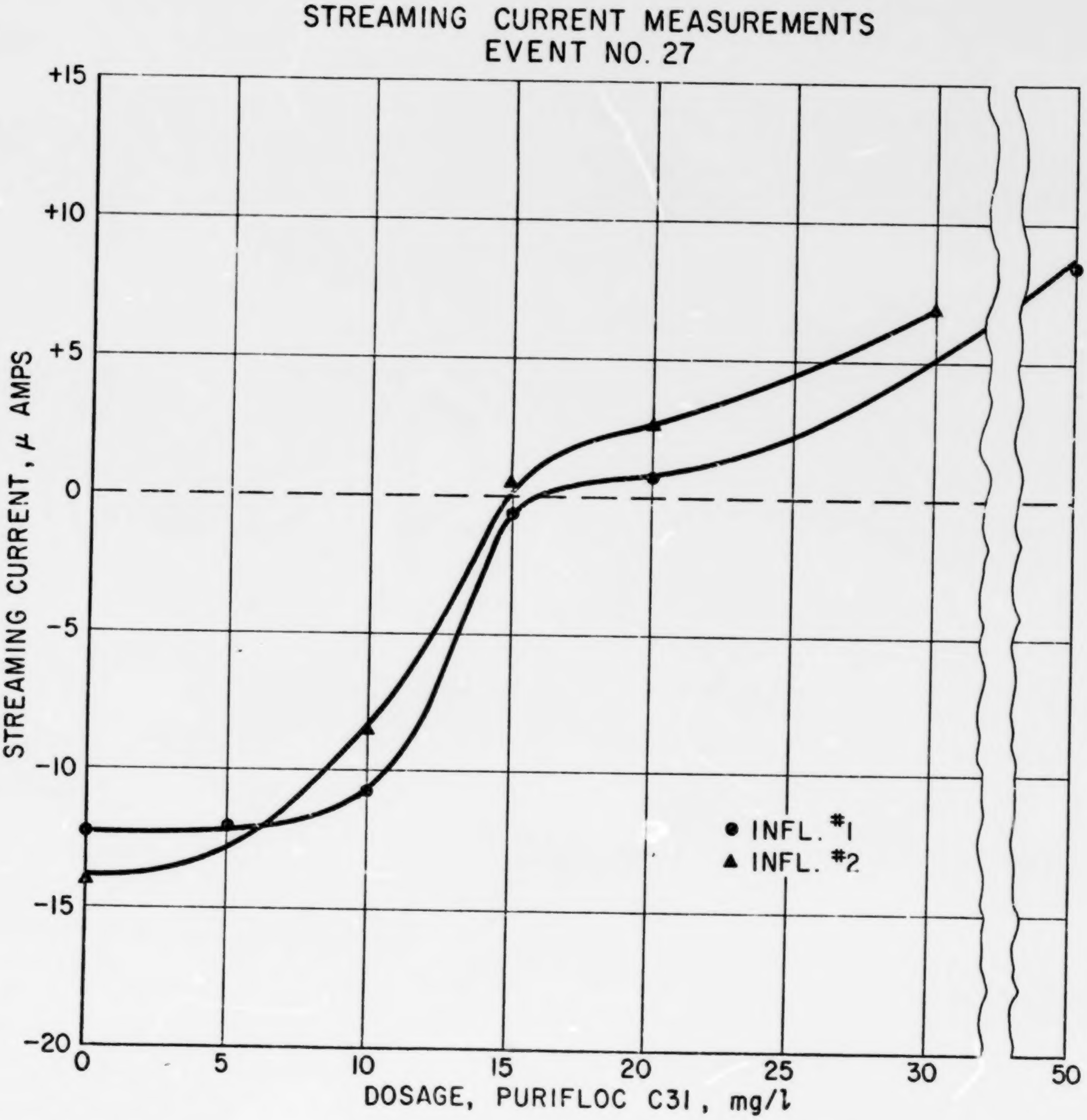
COMPARISON OF STREAMING CURRENT AND OTHER QUALITATIVE CRITERIA OF FLOCCULATION

Sample	Streaming Current (μ amps)	Floc Size	Clarity	Rating
Influent No. 1 (High solids, turbidity = 70 JTU)				
Blank	-12.2	None	P	5
5 mg/l C31	-12.0	PP	P	4
10 mg/l C31	-10.8	S	P	4
15 mg/l C31	- 0.6	M	F-P	2
20 mg/l C31	+ 0.8	M-L	F	1
50 mg/l C31	+ 8.4	S-M	F-P	3
Influent No. 2 (Low solids, turbidity = 35 JTU)				
Blank	-14.0	None	P	4
10 mg/l C31	- 8.4	S	P	3
15 mg/l C31	+ 0.5	S	P	3
20 mg/l C31	+ 2.5	S-M	P	2
30 mg/l C31	+ 7.0	S	P	3
15 mg/l Baroid Hectorite + 15 mg/l C31	+ 0.4	M	F	1

Key:

Floc Size	Clarity
PP = Pinpoint	P = Poor
S = Small	F = Fair
M = Medium	
L = Large	

Figure 26


STREAMING CURRENT MEASUREMENTS
EVENT NO. 27
+15
+10
+5
0
-5
-10
-15
-20
STREAMING CURRENT, μ AMPS
0
5
10
15
20
25
30
50
DOSAGE, PURIFLOC C31, mg/l
● INFL. #1
▲ INFL. #2

STABILITY OF PURIFLOC FLOCCULANTS

Over normal periods of storage, the PURIFLOC flocculants are quite stable. Periods of storage should not exceed one year to avoid loss of activity due to slight natural degradation of flocculant. This limitation is more important when storage temperatures are high or after a package or container has been opened. PURIFLOC flocculants can be adversely affected by temperatures over 48.8°C (120°F) either in bulk as dry powders or liquids, or in dilute solutions. Bulk powder or liquid products generally will not approach this temperature unless confined in unventilated spaces, or stored or utilized near steam lines or heating equipment.

The dry PURIFLOC flocculants are hygroscopic and will absorb moisture from the atmosphere. Opened bags should be stored only in low humidity environments and for short periods of time to prevent caking. The liquid PURIFLOC flocculants should be kept from freezing and preferably stored inside to prevent weathering of the containers and to reduce increases in viscosity due to lowering temperatures.

The recommended times of maximum storage for bulk PURIFLOC products and selected solutions are summarized in Table XXVI.

TABLE XXVI

RECOMMENDED MAXIMUM STORAGE TIMES OF
PURIFLOC FLOCCULANTS

Product	Bulk Product	Stock Solution Conc. %	Stock Solution Storage	Laboratory Working Soln. %
PURIFLOC N11	One year	3.0	1 month	0.1
PURIFLOC N12	One year	3.0	2 weeks	0.1
PURIFLOC A21	One year	3.0	1 month	0.1
PURIFLOC A22	One year	2.0	1 month	0.1
PURIFLOC A23	One year	0.5	1 month	0.05-0.1
PURIFLOC C31	One year	All	2-3 months	0.1 -10.0
PURIFLOC C32	One year	All	2-3 months	0.1 -10.0
ET-721	One year	5.0	1 month	0.1

The dry powders of PURIFLOC N11, N12, A21, A22, A23, and ET-721 flocculants should not be stored for longer than 12 months. Stock solutions are stable for a period of at least one month, with the exception of PURIFLOC N12. More dilute (0.05%) solutions should be prepared daily by dilution of the concentrated stock solutions. Liquid cationic flocculants are considered stable for 12 months or longer from time of manufacture. Dilute solutions of the cationic flocculants are stable for at least one month.

Some method of determining polymer degradation is necessary to express "storage stability" or "shelf life." The most easily measured parameter of degradation is the change in viscosity of a flocculant solution. Other parameters such as molecular weight (derived from viscosity measurements) and flocculant activity on selected substrates are also used. Unfortunately, viscosity does not completely characterize a flocculant without other parallel analyses to determine flocculant activity. A laboratory evaluation of flocculation activity is best for evaluating the storage stability of such materials, but there is no universal substrate that has adequate chemical and physical characteristics that can be used to satisfactorily screen all types of flocculants.

Caution should be exercised when relating viscosity to effectiveness as a flocculant. A material classed as a flocculant may have a specified level of viscosity and still have no activity as a flocculant. This can be due to contamination of the prime flocculant with materials which may cause gelation, precipitation, or other reactions which may or may not alter viscosity, but cause considerable loss of activity.

Under controlled laboratory conditions, viscosity as a parameter for studying stability or degradation of a flocculant material is accepted as an indication of what will occur in actual practice. Loss of viscosity under controlled conditions of storage can be attributed to flocculant instabilities.

The polyacrylamide-based materials are the most susceptible to degradation of the several PURIFLOC flocculants. PURIFLOC N12 is adversely affected when stored as a solution. It degrades to 50 percent of its original viscosity in two weeks. The dry product is the least stable of the products considered, retaining about 75 percent of its original activity after one year.

PURIFLOC flocculants N11, A22, and A23 also degrade under the same conditions as N12, but to a lesser extent. Solutions of PURIFLOC A22 degrade to about 83 percent of their original viscosity in two weeks. The dry powder will retain essentially all of its original activity for one year. PURIFLOC A23 has the same stability characteristics as PURIFLOC A22. The stability of solutions of PURIFLOC N11 is about the same as PURIFLOC N12. It is manfactured as a non-hydrolyzed polyacrylamide. In solution it will slowly hydrolyze into a form chemically similar to PURIFLOC N12. An activity of 93 percent of the original is retained after one year.

The viscosity of PURIFLOC A21 is not clearly related to its activity as a flocculant. A test to measure the efficiency of this flocculant consists of adding the flocculant to a concentrated aqueous slurry of MINCO BOND® clay, allowing floc to form, and timing the fall of the solids-liquid interface over a predetermined distance.

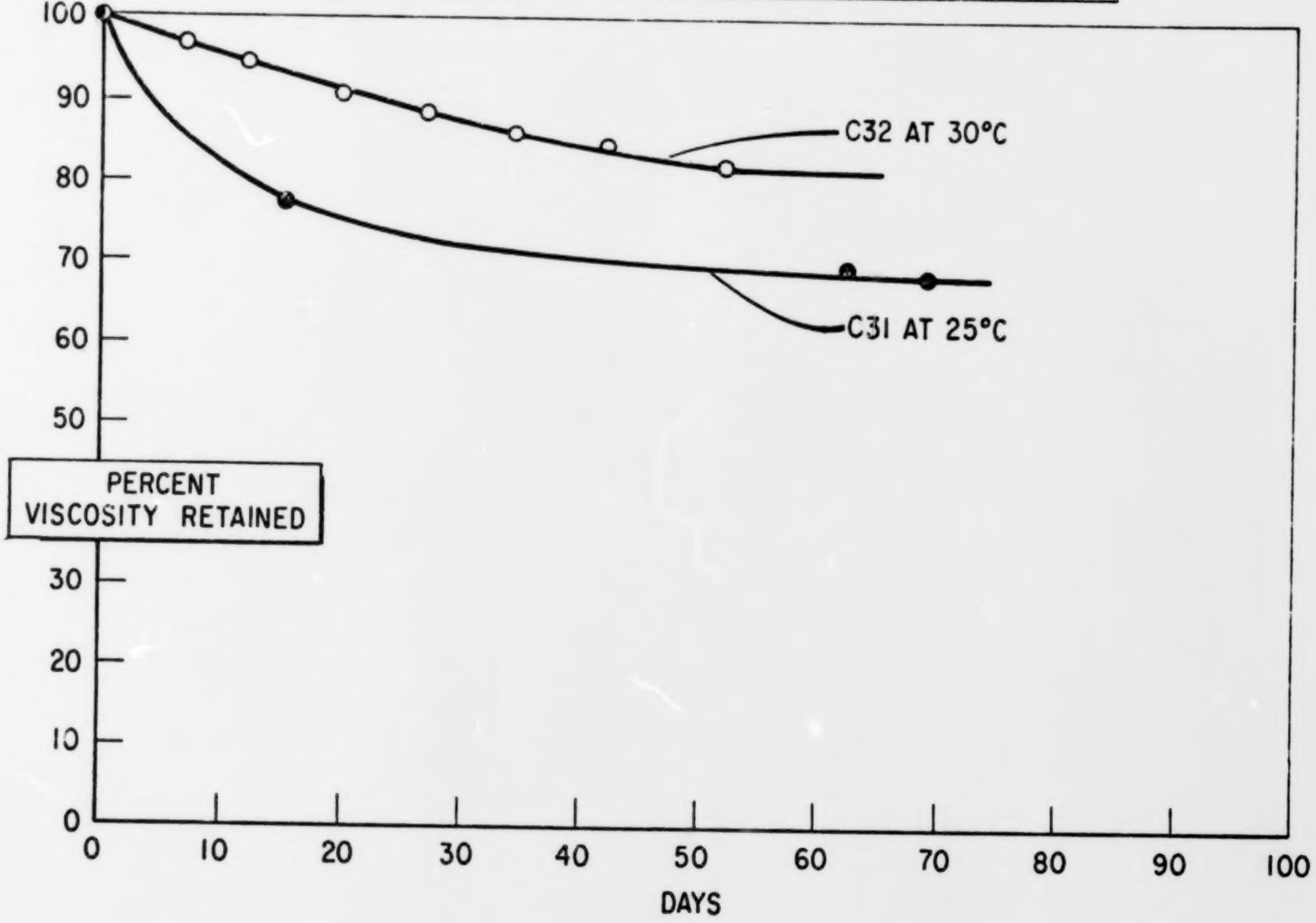
Dry PURIFLOC A21 degrades to about 65 percent of its original flocculant activity after one year of storage. A one percent solution degrades to less than 50 percent of its original activity in two weeks. Stability improves considerably as the concentration of the solution is increased. The maximum recommended concentration (3.0 percent) will remain nearly 100 percent active for at least one month.

At 30°C (86°F), PURIFLOC C32 retains about 80 percent of its original viscosity after three months storage. For the same storage time at 10°C, 90 percent viscosity is retained. About 65 percent of the original viscosity of diluted solutions of PURIFLOC C32 remains after three months storage at 30°C. The temperature stability of PURIFLOC C31 is similar to that of PURIFLOC C32. After three months bulk storage at room temperature, PURIFLOC C31 retains about 90 percent of its original viscosity. Dilute solutions of PURIFLOC C31 retain about 65 percent of their original viscosity after three months storage at 25°C. Reductions in viscosity of 1 percent solutions of PURIFLOC C31 and PURIFLOC C32 upon storage are shown in Figure 27.

Storage stability is necessary for design of flocculant dispersion and storage facilities. Liquid organic polymeric flocculants are similar to other common inorganic chemicals in respect to the types of storage tanks, piping and pump equipment used. Common materials of construction with the exceptions of zinc or aluminum and their alloys can be used for handling the organic flocculants. Solutions of organic flocculants can tolerate metal ions such as calcium, magnesium, and iron at concentrations normally found in water systems used in preparing and diluting stock and working solutions of flocculants. An excess of certain metal ions developed as corrosion products in the flocculant system can reduce flocculant activity. Trivalent cations, certain divalent cations, and some high molecular weight organic cations react with anionic flocculants.

Figure 27

PERCENT VISCOSITY RETAINED (1% SOLUTION)



Bulk handling and dissolution of the flocculants require specialized equipment. Excessive shear rates, extreme temperatures, and make-up water of poor quality should be avoided. The minimum dissolution time ordinarily required for maximum concentrations are on the order of 10 minutes. Mixing and detention time can become critical in sizing tanks for large scale operations. Efforts to reduce the time of dissolution by using high-shear-rate mixing equipment usually leads to degradation of flocculant by chopping the long polymer molecules into shorter, less effective chains. High-shear equipment includes propeller type mixers exceeding 350 rpm in tanks of moderate volume (300 gallons). High shear rates may also occur in piping designs with many 90° bends and other obstructions. Long sweeping bends or hoses are desirable to minimize localized high flow velocity points. Air spargers can be installed to provide agitation in lieu of propeller mixers.

TOXICITIES OF PURIFLOC FLOCCULANTS

In addition to obtaining effective flocculation, there are two other important criteria for a combined sewage treatment utilizing polyelectrolyte flocculants.

1. The flocculants used must be relatively non-toxic to humans, animals and aquatic organisms.
2. The effective shelf life of the dry flocculant and its solution must be sufficient to prevent a significant loss of treatment activity between events.

Following are the results of studies conducted by The Dow Chemical Company relating to the toxicity and storage of the polymeric flocculants used in the Milk River Combined Sewer Overflow Project. These data are specific to the flocculants listed in Table XXVII.

Fish Toxicity

Flocculants are generally classified as to their ionic nature, functional group or chemical structure. The PURIFLOC series can be divided, from the standpoint of toxicity, into polyacrylamides, SPSS, and polyamines, with the functional groups $-NH_2$ and $-COO^-$, $-SO_3^-$, and $-NR_3^+$ respectively.

Nitrogenous cationic flocculants apparently can limit the ability of fish to obtain oxygen from the water. These flocculants apparently flocculate the secretion from the gills of the fish, thereby coating the oxygen-absorbing membranes and reducing the transfer of oxygen. The starting monomer for PURIFLOC C31 has a fish toxicity maximum safe

TABLE XXVII

PURIFLOC FLOCCULANTS EVALUATED FOR TOXICITY AND STORAGE

Flocculant	Description
PURIFLOC N17	polyacrylamide - potable water grade
PURIFLOC N11	polyacrylamide - minimum hydrolysis
PURIFLOC N12	polyacrylamide - low hydrolysis
PURIFLOC A21	sodium polystyrene sulfonate
PURIFLOC A22	polyacrylamide - 30% hydrolysis
PURIFLOC A23	high molecular weight polyacrylamide - 25% hydrolysis
PURIFLOC C31	polyalkylene polyamine
PURIFLOC C32	polyethylenimine
ET-721 (developmental)	low hydrolysis polyacrylamide made partially cationic with dimethylamine via the Mannich reaction

limit in excess of 100 mg/l. After polymerization the same chemical, due to its resulting flocculant characteristics, has a fish toxicity maximum safe limit of 2 mg/l of polymer in solution.

The polyacrylamide flocculants dissolved in water in gross amounts are toxic to fish but in a different manner than the polyamines. The polymer is not chemically toxic, but the increased viscosity of the water caused by the polymer decreases the mobility of the fish thereby causing suffocation. The maximum safe limit of acrylamide monomer is greater than 100 mg/l which is essentially the same as the polyamines.

Fish toxicities were compiled by the Midland Division Waste Control of The Dow Chemical Company and listed in Table XXVIII. This table is a guideline for situations in which there may be residual polymer in flocculant-treated waters.

As can be seen from Table XXVIII, the maximum safe limits of polyacrylamide-based anionic flocculants are from 500 to 1000 mg/l. These high residuals seldom if ever occur in normal flocculation practice. With the exception of PURIFLOC A23, 1000 mg/l of polyacrylamide-based flocculants can be tolerated by fathead minnows without adverse effect.[4] A concentration of 2500 mg/l of PURIFLOC A22 was 100% fatal[4] Sodium polystyrene sulfonate (PURIFLOC A21) also is quite viscous in solution and apparently has the same effect on minnows as polyacrylamide.

TABLE XXVIII

FISH TOXICITIES OF PURIFLOC FLOCCULANTS*

	mg/l in Solution		
Polymer	Maximum Safe	Partial Kill	100% Fatal
PURIFLOC N17	1000		
PURIFLOC N11	1000		
PURIFLOC N12	1000		
PURIFLOC A21	1000		
PURIFLOC A22	1000	--	2500
PURIFLOC A23	500	--	1000
PURIFLOC C31	2.0	3.0	4.0
PURIFLOC C32	2.0	3.0	5.0
ET-721	2.0	3.0	10.0

*Acute toxicity to fathead minnows, *Pimephales promelas*, in Lake Huron water at 50°F after 72 hours.

The amounts of flocculant that can be tolerated in a receiving stream can be increased if there is sufficient turbidity in the form of suspended solids. Excess flocculant can be adsorbed onto the solids and removed from the water. Cationic flocculants of the type discussed in this report are readily adsorbed on substrates such as various types of clay, river silt, charcoal, $CaCO_3$, and other materials which possess anionic surface charges. The degree of adsorption is dependent on the number of sites available on the substrate, the molecular structure of the polymer, the pH and alkalinity of the system, and other parameters.

The maximum safe limit of PURIFLOC C31 could be increased from 2 mg/l to 15 and 25 mg/l by addition of 50 and 100 mg/l of bentonite clay, respectively. The maximum safe limit of 2 mg/l in the case of PURIFLOC C32 was increased to 5-10 mg/l and 10 mg/l by addition of 50 and 100 mg/l of bentonite clay, respectively. The acrylamide-based flocculants are also readily adsorbed on suspended solids providing conditions conducive to flocculation exist[5].

A recent study by The Dow Chemical Company has shown that cationic polymers are effectively removed from river water and raw sewage by adsorption on suspended solids. It was found that maximum adsorption of flocculants on suspended solids occurred during flocculation. Fish toxicities of the cationic polymers must be based on the amount of polymer remaining in solution after flocculation and not on the total concentration initially applied.

Human and Animal Toxicity

All of the PURIFLOC flocculants listed above are low in oral toxicity and low in degree of health hazard from handling. None of the polymeric flocculants noted are absorbed through the skin in toxic amounts. Prolonged skin contact results in no significant irritation although repeated or prolonged gross contact may result in mild contact dermatitis. The effects of eye contact may range from essentially no effect to mild transitory irritation, depending on the flocculant. In general, human experience has shown that these products are safe to handle but reasonable care and personal cleanliness should be practiced to avoid possible skin and eye contact.

Table XXIX contains LD_{50} values for laboratory rats exposed to the PURIFLOC series. These data were obtained from reports issued by the Biochemical Research Laboratory of The Dow Chemical Company. Human LD_{50} values can be estimated by multiplying the LD_{50} value for rats by the human body weight in kilograms.

TABLE XXIX

LD_{50} VALUES FOR LABORATORY-CONFINED RATS

Flocculant	LD_{50} (grams/kilogram)
PURIFLOC N17	4.0 +
PURIFLOC N11	4.0 +
PURIFLOC N12	4.0 +
PURIFLOC A21	2.0
PURIFLOC A22	4.0 +
PURIFLOC A23	4.0 +
PURIFLOC C31	1.0 - 2.0
PURIFLOC C32	1.1
ET-721	4.0

Specifications for the manufacture of polyacrylamide include a maximum acrylamide monomer content of <0.05 percent on a product basis. This is a safe limit for potable water uses and is within the Public Health Service recommendations. Sodium styrene sulfonate monomer is not toxic, but forms aldehydes after prolonged storage which are slightly irritating to skin and nose membranes. The presence of monomer in polyamine polymers is insignificant due to the reactivity of these compounds in polymerization. Essentially no detectable monomer concentration remains.

In the preparation of ET-721, a nonionic acrylamide is made cationic via aminomethylation (Mannich reaction). The aminomethylating process depends upon the reaction of dimethylamine (DMA) with acrylamide via an intermediate, which gives the polymer its cationic character. DMA as a gas, or dissolved in water, is toxic. In the preparation of ET-721, the DMA reacts completely to form part of the polymer molecule thereby eliminating its availability as the parent compound. The finished polymers are as non-toxic as the polyamines PURIFLOC C31 and C32.

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SECTION 7

LONG-TUBE SEDIMENTATION STUDIES

SEDIMENTATION

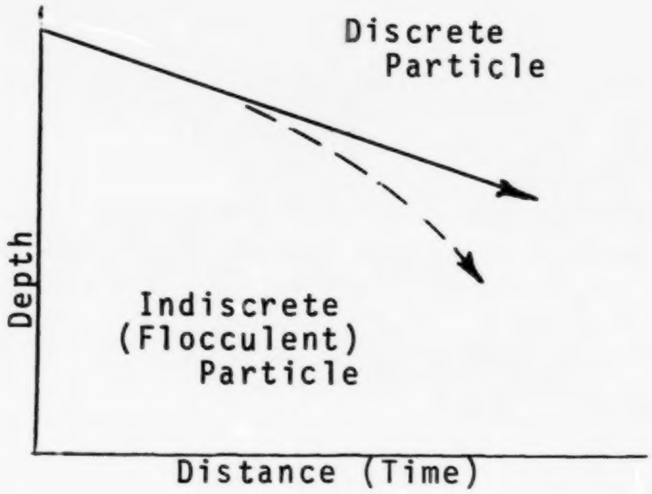
"Sedimentation is a waste treatment process whereby suspended and coagulated particles of a density greater than that of the liquid medium are removed."[1] Under quiescent conditions, the factors influencing sedimentation of discrete suspended particles are their size, shape, and density. The viscosity and density of the suspending medium may also require consideration. The settling velocity of a discrete particle can be defined on the bases of these characteristics and is assumed constant during the sedimentation process.

Complications may arise in the case of sedimentation of suspended particles from combined sewer overflows. The particles are essentially non-spherical, polydisperse, and possess variable degrees of surface charge. They also may be of a flocculent nature, i.e., increasing in size and density, and changing in shape during sedimentation. The settling velocity of such indiscrete* particles is not constant.

The settling path of a discrete particle is linear in an ideal tank. The corresponding path of an indiscrete* (flocculent) particle is curvilinear. Discrete solids are denoted as Class I solids; indiscrete solids are denoted as Class II solids. The settling paths of these two classes of particles are portrayed in Figure 28.

Figure 28

Settling Paths of Discrete and Indiscrete Particles



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*Not to be confused with indiscreet, i.e., lacking prudence, which could also apply to certain exasperating experimental situations.

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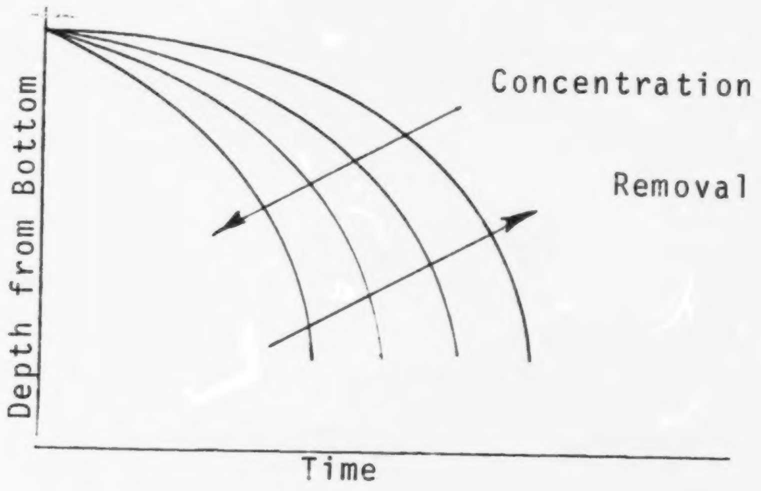
The extent of discreteness must first be determined before the efficiency of removal can be predicted for a given system. Removal of discrete particles is dependent only on the settling velocity (or overflow rate). Removal of indiscrete particles is a function of both settling velocity and detention time.

Laboratory experiments are generally advisable to evaluate sedimentation for an unknown suspension. These usually consist of measuring the concentration of suspended solids in a sedimenting fluid at various depth increments and time intervals in a vertical cylinder. The initial profile of solids concentration must be uniform. The concentration of solids in the upper regions of the suspension thereafter decreases as sedimentation proceeds. Limiting (or maximum) sedimentation contours of concentration or of removal as functions of depth and time are represented in Figure 29.

Figure 29

Sedimentation Contour Map: Depth from Bottom vs. Time

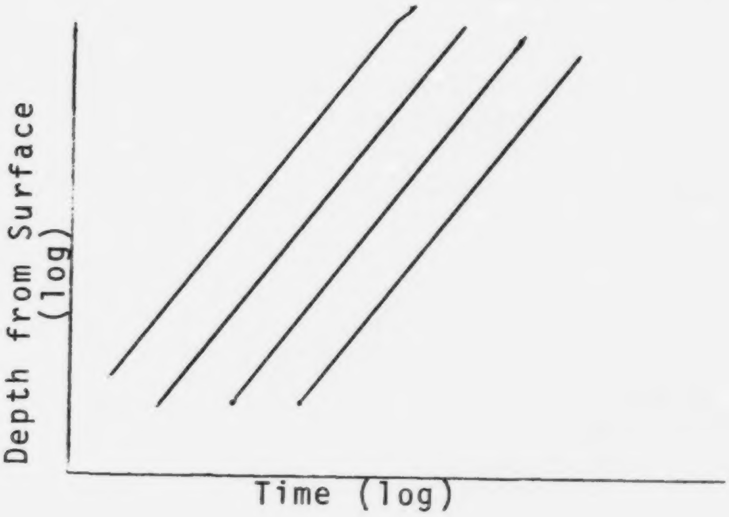


Such contour plots are useful in determining the extent of removal of solids from an ideal settling tank at a given depth-time coordinate.

A measure of the discreteness of a given suspension can be determined graphically by assuming an exponential variation of settling velocity and detention time[5]. The previously described contours form a family of parallel lines in a logarithmic plot of depth vs. time as shown in Figure 30.

Figure 30

Sedimentation Contour Map:
Log Depth from Surface vs. Log Time

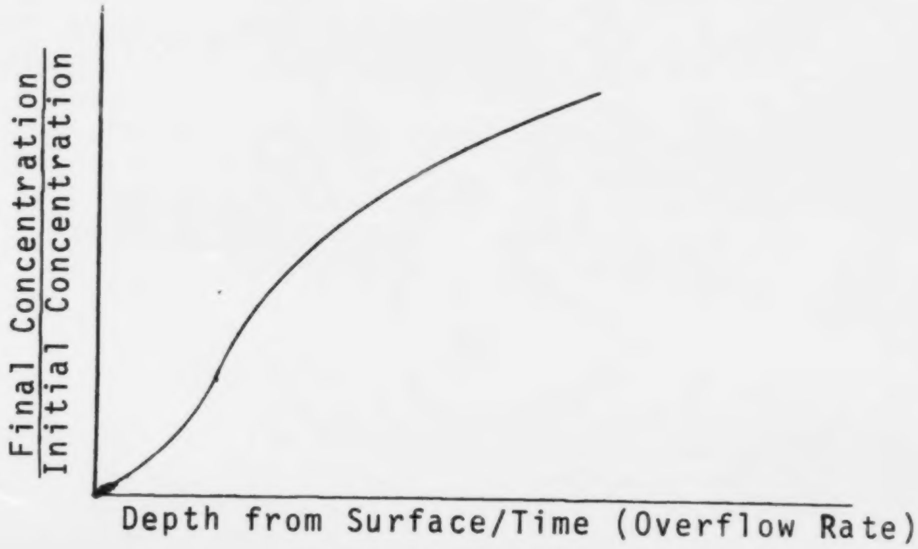


A slope of unity implies discrete sedimentation. The slope exceeds unity as the flocculent nature of the particles increases. The preceding contour diagrams are informative but final design curves require sectioning at a selected time, computing settling velocities (depth/time), and plotting removal vs. overflow rate.

A further development is the reduced concentration profile[4]. All concentration data in the depth-time grid are normalized as fractions of the initial depth-average concentration and plotted against the quotient of depth/time as shown in Figure 31.

Figure 31

Reduced Concentration Profile: Fraction Initial
Depth-Average Concentration vs. Overflow Rate

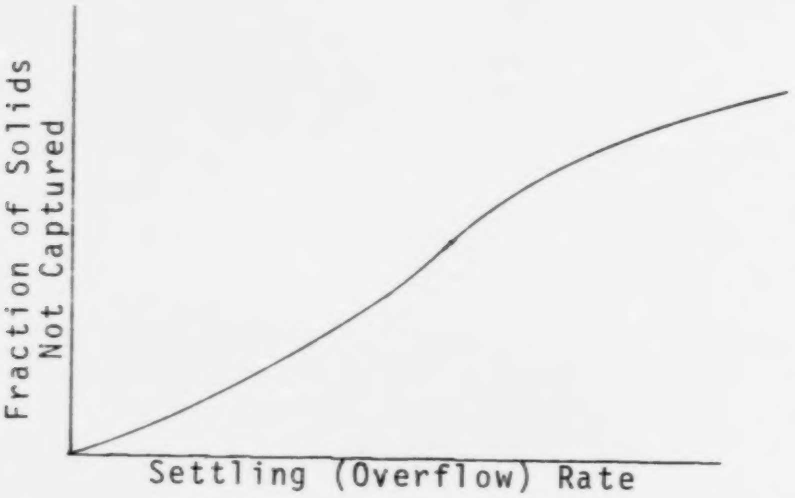


The depth/time quotient corresponds dimensionally to a settling or overflow rate (L/T)*. If indeed the system under consideration is Class I (discrete) then all data should plot along a single profile in such a graph. The spread of the data is, therefore, a measure of the deviation from discreteness.

The final step in the analysis of the sedimentation process is to compute the area under the reduced concentration profile at selected overflow rates. The fraction of solids not captured at a given overflow is equal to the corresponding cumulative area. The fractional loss profile is illustrated in Figure 32.

Figure 32

Fractional Loss Profile:
Fraction of Solids not Captured vs. Overflow Rate



The average detention times at selected stations across an effluent weir of a settling basin can be determined from dye dispersion studies[3,7]. The local overflow rate at each station (basin depth-average detention time) is then used to determine the predicted local value of the fraction of solids not captured and ultimately an average fraction for the entire basin.

Five systems of flocculants as established by the flocculant studies were evaluated using a long-tube sedimentation device. A description of this device, its operation, and performance is contained within this chapter. Removals of optical solids were predicted for each system for selected overflow rates and combined with results from the hydraulic model studies for design purposes.

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*This quantity can be reported as a velocity, L/T, or surface loading (L^3/TL^2). It is not a weir overflow rate.

DEVELOPMENT OF A LONG-TUBE SEDIMENTATION DEVICE

A usable long-tube sedimentation device (LTSD) is a requisite for studying sedimentation processes[4,5]. The dimensions of such a device should be sufficient to approximate full-scale conditions and to minimize wall effects.

The long-tube sedimentation device used in these studies was made of a clear plastic cylinder having a wall thickness of 1/4 in., an inside diameter of 5 in., and a length of 12 ft. A schematic of the LTSD is presented in Figure 33. The tube contained 50 liters of liquid when completely filled. It was fitted with stainless steel paddles with one-inch square blades placed vertically at 4 in. intervals along a central drive shaft. Sampling ports were located at one-foot intervals and were covered with large syringe caps. The holding tank has a capacity of 35 gallons, sufficient for two 50 liter charges to the LTSD plus associated jar tests. Sample charges were transferred to the column by a centrifugal pump.

Samples of 100 ml volume were collected by hypodermic syringe at preselected depths and times. A sample grid of six depths (1, 2, 4, 6, 8, and 10 feet from bottom) and eight times (0, 4, 9, 16, 25, 36, 49, and 64 minutes) was established for the majority of the systems studied. A sludge sample was collected at the end of 64 minutes at a depth of 0 feet.

Optical solids were rapidly determined[1,2,6] using a photometric probe that could be immersed directly into each sample. Optical solids were defined as absorbance x 1000. Gravimetric solids were determined after filtration through Gooch crucibles and subsequent drying to constant weight. A typical correlation is shown in Figure 34.

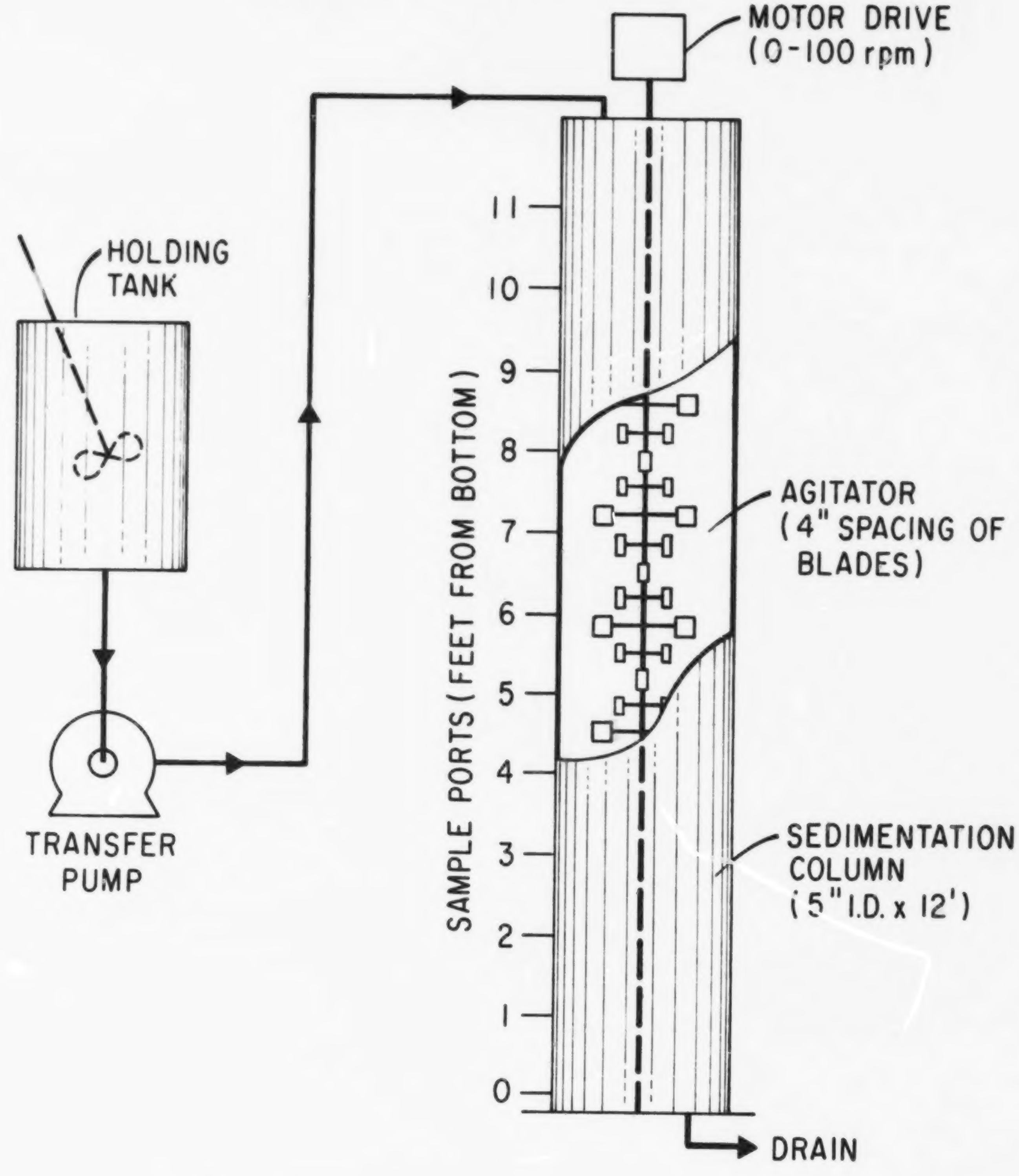
PROCESSING OF SOLIDS CONCENTRATION DATA

The raw data of optical and gravimetric solids concentrations as functions of depth and time must be processed prior to graphical analysis. An IBM 1130 computer was used for both processing and plotting[8].

Withdrawing of each sample reduces the total volume of the system. The depth coordinate is corrected for this depletion. A finite time is required to withdraw each sample. The time coordinate is corrected accordingly.

Figure 33

LONG-TUBE SEDIMENTATION DEVICE

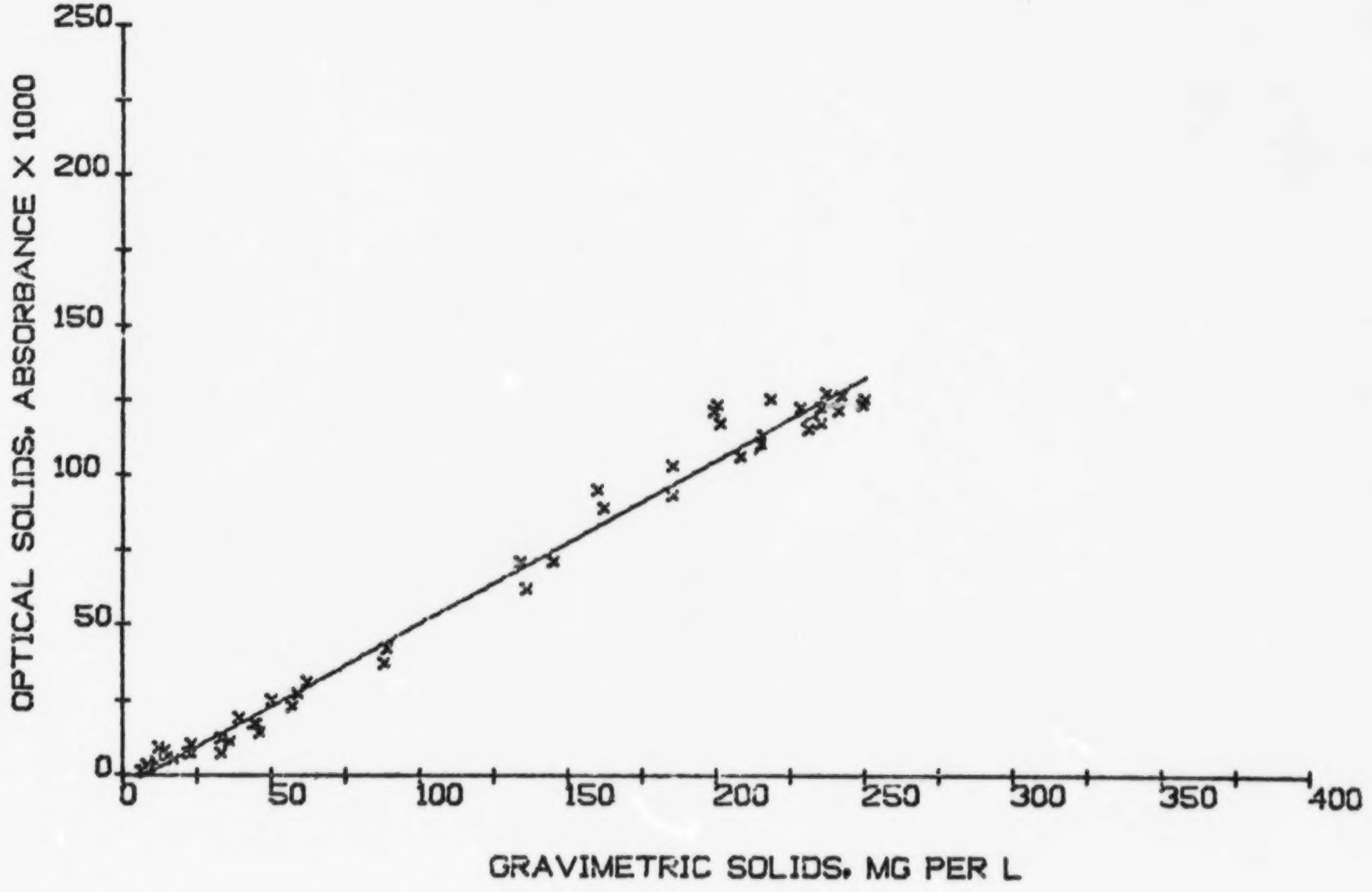

MOTOR DRIVE (0-100 rpm)
HOLDING TANK
AGITATOR (4" SPACING OF BLADES)
TRANSFER PUMP
SAMPLE PORTS (FEET FROM BOTTOM)
11
10
9
8
7
6
5
4
3
2
1
0
SEDIMENTATION COLUMN (5" I.D. x 12')
DRAIN

SAMPLING { DEPTHS: (0), 1, 2, 4, 6, 8, 10 FT.
TIMES: 0, 4, 9, 16, 25, 36, 49, 64, MIN.

Figure 34

MILK RIVER PROJECT - LONG TUBE SEDIMENTATION STUDY
CORRELATION OF OPTICAL AND GRAVIMETRIC SOLIDS

SYSTEM 5A
STORM FLOW 11-18-63
AVG INIT OPTICAL SOLIDS = 120 (A X 1000)
AVG INIT GRAVIMETRIC SOLIDS = 206 MG PER L
20 MG PER L PURIFLOC C31 ÷ 20 MG PER L BAROID HECTORITE CLAY



Removals are calculated from the concentration data and reported as fractions. The coordinates of the reduced profile are also machine calculated. The resulting corrected data and computed parameters are then punched on cards for subsequent graphical analysis.

A typical grid of values of optical solids as functions of uncorrected depth from bottom and uncorrected time of sampling is presented in Table XXX.* The initial (time zero) depth-averaged concentrations of optical and gravimetric solids, respectively, were 119.5 and 206.2. The initial profile at time zero is uniform. Concentrations decrease with time as the solids settle to the bottom of the column.

The data corrected for volume depletion and sampling delay are presented in Table XXXI. Other quantities have been calculated for use in the graphical presentations. Removals of optical and of gravimetric solids are calculated as fractions of the initial depth-averaged concentrations. The settling rate is calculated by dividing the depth from the surface by the time of sampling.

VARIABLES

Dry weather flow was the flocculate used during the "shakedown" of the LTSD and associated sampling procedures. Samples of combined sewer overflow taken during the first pumping were used to evaluate all subsequent flocculant systems. The flocculants used were: PURIFLOC C31, and/or PURIFLOC A23. The coagulant aids used were: ferric chloride and/or BAROID HECTORITE clay. Optimum concentrations of all chemicals to be used in subsequent LTSD studies were determined on the basis of jar tests.

Flocculation was initiated in the early studies by injecting incremental portions of solutions of the selected flocculants by syringe through the sampling ports of the LTSD. This procedure proved unsatisfactory due to poor dispersion. All subsequent flocculations were conducted by flash mixing a single total dose of the flocculant in a full-tube charge of stormwater (50 liters) and then pumping the mixture to the LTSD.

Sedimentation under quiescent conditions resulted in deposition of flocculent solids on the agitator blades. This deposition produced skewed profiles of solids concentrations. A condition of dynamic sedimentation was established in all subsequent

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*Nomenclature provided at end of report.

TABLE XXX

11-18-69 20 MG PER L PURIFLOC C31 + 20 MG PER L BAROID HECTORITE CLAY

RAW DATA - SAMPLING TIMES AND DEPTHS ARE UNCORRECTED

DEPTH, FT	OPTICAL SOLIDS (ABSORBANCE X 1000)							
TIME, MIN	0.	4.	9.	16.	25.	36.	49.	64.
1.	123.	124.	127.	126.	90.	18.	11.	8.
2.	124.	123.	128.	114.	72.	20.	8.	5.
4.	122.	122.	116.	96.	43.	15.	7.	4.
6.	126.	118.	107.	63.	32.	13.	6.	2.
8.	118.	111.	72.	28.	18.	12.	10.	2.
10.	104.	94.	38.	24.	26.	8.	9.	3.

DEPTH, FT	GRAVIMETRIC SOLIDS (MG/L)							
TIME, MIN	0.	4.	9.	16.	25.	36.	49.	64.
1.	229.	250.	243.	251.	163.	45.	24.	24.
2.	201.	236.	238.	217.	135.	40.	23.	12.
4.	200.	242.	232.	161.	90.	47.	16.	9.
6.	219.	236.	209.	137.	63.	34.	18.	8.
8.	202.	216.	146.	60.	46.	37.	13.	7.
10.	186.	186.	89.	58.	51.	34.	15.	10.

INITIAL AND CALCULATED CONSTANTS

P = 6
Q = 8
DO = 142.75 (IN)
DF = 127.00 (IN)
TSP = 3.0 (MIN)

OSO = 119.500 (A X 1000)
GSO = 206.166 (MG/L)
TDEPL = 0.02440 (FT/SPL)
ADEPL = 0.02734 (FT/SPL)

TABLE XXXI

11-18-69 20 MG PER L PURIFLOC C31 + 20 MG PER L BAROID HECTORITE CLAY

CORRECTED DATA AND CALCULATED PARAMETERS

N	M	D(M)	DBOT	DSUR	LDSUR	T(N)	TSPL	LTSPL	OS	GS	ROS	RGS	SETR
1	1	1.	1.000	10.895	1.037	0.	0.000		123.	229.	-0.029	-0.110	∞
1	2	2.	2.000	9.895	0.995	0.	0.00C		124.	201.	-0.037	0.025	∞
1	3	4.	4.000	7.895	0.897	0.	0.000		122.	200.	-0.020	0.029	∞
1	4	6.	6.000	5.895	0.770	0.	0.000		126.	219.	-0.054	-0.062	∞
1	5	8.	8.000	3.895	0.590	0.	0.000		118.	202.	0.012	0.020	∞
1	6	10.	10.000	1.895	0.277	0.	0.000		104.	186.	0.129	0.097	∞
2	1	1.	1.000	10.731	1.030	4.	0.875	-0.057	124.	250.	-0.037	-0.212	2.044
2	2	2.	1.972	9.759	0.989	4.	0.791	-0.101	123.	236.	-0.029	-0.144	2.054
2	3	4.	3.945	7.786	0.891	4.	0.708	-0.149	122.	242.	-0.020	-0.173	1.832
2	4	6.	5.917	5.813	0.764	4.	0.625	-0.204	118.	236.	0.012	-0.144	1.550
2	5	8.	7.890	3.841	0.584	4.	0.541	-0.266	111.	216.	0.071	-0.047	1.181
2	6	10.	9.863	1.868	0.271	4.	0.458	-0.338	94.	186.	0.213	0.097	0.679
3	1	1.	1.000	10.567	1.023	9.	1.708	0.232	127.	243.	-0.062	-0.178	1.030
3	2	2.	1.945	9.622	0.983	9.	1.625	0.210	128.	238.	-0.071	-0.154	0.986
3	3	4.	3.890	7.677	0.885	9.	1.541	0.187	116.	232.	0.029	-0.125	0.829
3	4	6.	5.835	5.731	C.758	9.	1.458	0.163	107.	209.	0.104	-0.013	0.655
3	5	8.	7.781	3.786	0.578	9.	1.375	0.138	72.	146.	0.397	0.291	C.458
3	6	10.	9.726	1.841	0.265	9.	1.291	0.111	38.	89.	0.682	0.568	0.237
4	1	1.	1.000	10.403	1.017	16.	2.875	0.458	126.	251.	-0.054	-0.217	0.603
4	2	2.	1.917	9.485	0.976	16.	2.791	0.445	114.	217.	0.046	-0.052	0.566
4	3	4.	3.835	7.567	0.878	16.	2.708	0.432	96.	161.	0.196	0.219	0.465
4	4	6.	5.753	5.649	0.751	16.	2.625	0.419	63.	137.	0.472	0.335	0.358
4	5	8.	7.671	3.731	0.571	16.	2.541	0.405	28.	60.	0.765	0.708	0.244
4	6	10.	9.589	1.813	0.258	16.	2.458	0.390	24.	58.	0.799	0.718	0.122
5	1	1.	1.000	10.239	1.010	25.	4.375	0.640	90.	163.	0.246	0.209	0.390
5	2	2.	1.890	9.348	0.970	25.	4.291	0.632	72.	135.	0.397	0.345	0.363
5	3	4.	3.781	7.458	0.872	25.	4.208	0.623	43.	90.	0.640	0.563	0.295
5	4	6.	5.671	5.567	0.745	25.	4.125	0.615	32.	63.	0.732	0.694	0.224
5	5	8.	7.562	3.677	0.565	25.	4.041	0.606	18.	46.	0.849	0.776	0.151
5	6	10.	9.453	1.786	0.251	25.	3.958	0.597	26.	51.	0.782	0.752	0.075
6	1	1.	1.000	10.075	1.003	36.	6.208	0.792	18.	45.	0.849	0.781	0.270
6	2	2.	1.863	9.212	0.964	36.	6.125	0.786	20.	40.	0.832	0.805	0.250
6	3	4.	3.726	7.348	0.866	36.	6.041	0.781	15.	47.	0.874	0.772	0.202
6	4	6.	5.589	5.485	0.739	36.	5.958	0.774	13.	34.	0.891	0.835	0.153
6	5	8.	7.453	3.622	0.558	36.	5.875	0.768	12.	37.	0.899	0.820	0.102
6	6	10.	9.316	1.759	0.245	36.	5.791	C.762	8.	34.	0.933	0.835	0.050
7	1	1.	1.000	9.911	0.995	49.	8.375	0.922	11.	24.	0.907	0.883	0.197
7	2	2.	1.835	9.075	0.957	49.	8.291	0.918	8.	23.	0.933	0.888	0.182
7	3	4.	3.671	7.239	0.859	49.	8.208	0.914	7.	16.	0.941	0.922	0.146
7	4	6.	5.507	5.403	0.732	49.	8.125	0.909	6.	18.	0.949	0.912	0.110
7	5	8.	7.343	3.567	0.552	49.	8.041	0.905	10.	13.	0.916	0.936	0.073
7	6	10.	9.179	1.731	0.238	49.	7.958	0.900	9.	15.	0.924	0.927	0.036
8	1	1.	1.000	9.747	0.988	64.	10.875	1.036	8.	24.	0.933	0.883	0.149
8	2	2.	1.808	8.938	0.951	64.	10.791	1.032	5.	12.	0.958	0.941	0.138
8	3	4.	3.617	7.130	0.852	64.	10.708	1.029	4.	9.	0.966	0.956	0.110
8	4	6.	5.425	5.321	0.725	64.	10.625	1.026	2.	8.	0.983	0.961	0.083
8	5	8.	7.234	3.513	0.545	64.	10.541	1.022	2.	7.	0.983	0.966	0.055
8	6	10.	9.042	1.704	0.231	64.	10.458	1.019	3.	10.	0.974	0.951	0.027

studies. This condition consisted of operating the agitator at 5 rpm during sedimentation to prevent deposition of solids. This dynamic condition is also an approximation to that existing for gravity sedimentation during conventional sewage treatment.

FLOCCULANT SYSTEMS

Five flocculant systems were evaluated using samples of combined sewer overflow. These systems are summarized in Table XXXII. Sedimentations of controls not treated with flocculants are also reported for comparison. Initial and final values of optical and gravimetric solids are averaged for samples at all depths except those containing the sludge layer.

System 1

The initial "shake-down" of the LTSD and associated procedures was conducted on the dry-weather flow of 6-5-69. The ternary system of BAROID HECTORITE clay + PURIFLOC C31 + PURIFLOC A23 in the ratios 20:20:0.75 mg/l was very effective. Dispersion of the chemicals, however, was not uniform when applied by series injection to the long-tube. All mechanical systems and physical analyses were satisfactory. The same ternary system described above was evaluated on combined sewer overflow from Event 27 on 6-5-69. The same concentrations of chemicals were used. The flocculation and subsequent sedimentation resulted in very acceptable solids removal and an exceptionally clear supernate.

System 2

A system of 20 mg/l PURIFLOC C31 was evaluated on stormwater for the second pumping of Event 29. Flocculation in the initial holding tank and subsequent transfer to the long-tube sedimentation device was much more effective than direct introduction of the flocculant to the tube. Tank flocculation was used in all subsequent studies.

System 3

A dual system of PURIFLOC C31 and PURIFLOC A23 was evaluated on the storm flow of 8-1-69. Optimum doses of the two chemicals as determined by jar tests were: 30 mg/l PURIFLOC C31 and 1 mg/l PURIFLOC A23. In addition to the control, sedimentation was evaluated under dynamic conditions of 5 rpm agitation and quiescent sedimentation at 0 rpm. Solids deposition on the paddles at 0 rpm resulted in local anomalies in the solids concentration contours. Dynamic sedimentation at the 5 rpm condition was established in all subsequent evaluations.

TABLE XXXII

COMPARISON OF REMOVAL EFFICIENCIES OF FIVE FLOCCULANT SYSTEMS BASED UPON INITIAL AND FINAL TIME AVERAGES OVER ALL DEPTHS FOR OPTICAL AND GRAVIMETRIC SOLIDS

System	Date	Flocculants (mg/l)	Optical Solids① Initial A_o	Final A_{64}	Removal $\left(\frac{A_o - A_{64}}{A_o}\right)$	Gravimetric Solids② Initial C_o	Final C_{64}	Removal $\left(\frac{C_o - C_{64}}{C_o}\right)$	Comments
1	6-5-69	None	$81.\overline{4}$	$62.\overline{0}$	$.23\overline{8}$	$79.\overline{4}$	$56.\overline{8}$	$.28\overline{5}$	
1A④	"	20 Clay + 20 C31 + .75 A23	$88.\overline{8}$	$5.\overline{8}$③	$.93\overline{5}$	$90.\overline{6}$	$5.\overline{8}$③	$.93\overline{6}$	
1B④	"		$69.\overline{6}$	$4.\overline{4}$	$.93\overline{7}$	$67.\overline{4}$	$5.\overline{3}$	$.92\overline{1}$	Dry weather flow
2	6-20-69	None	$96.\overline{6}$	$59.\overline{1}$	$.38\overline{8}$	80.7	$59.\overline{8}$	$.29\overline{9}$	Flocculation in
2A	"	20 C31	$84.\overline{8}$	$48.\overline{5}$	$.42\overline{8}$	$87.\overline{2}$	$23.\overline{0}$	$.73\overline{6}$	Column
2B	"	"	$55.\overline{7}$	$15.\overline{3}$	$.72\overline{4}$	$84.\overline{3}$	$5.\overline{3}$	$.93\overline{7}$	Tank
3	8-1-69	None	$110.\overline{2}$	$77.\overline{0}$	$.30\overline{1}$	$121.\overline{2}$	$37.\overline{0}$	$.69\overline{5}$	Sedimentation at
3A	"	30 C31 + 1 A23	$75.\overline{3}$	$2.\overline{7}$	$.96\overline{4}$	$187.\overline{7}$	$8.\overline{3}$	$.95\overline{6}$	0 rpm
3B	"	"	$120.\overline{5}$	$17.\overline{5}$	$.85\overline{5}$	$192.\overline{5}$	$18.\overline{0}$	.906	5 rpm
4	8-16-69	None	$123.\overline{8}$	$48.\overline{0}$	$.61\overline{2}$	$212.\overline{8}$	$43.\overline{8}$	$.79\overline{4}$	
4A	"	30 Fe + .5 A23	$126.\overline{3}$	$16.\overline{0}$	$.87\overline{3}$	$299.\overline{0}$	$17.\overline{0}$	.943	
5	11-18-69	Control	$149.\overline{0}$	$87.\overline{3}$	$.41\overline{4}$	$182.\overline{7}$	$83.\overline{5}$	$.54\overline{3}$	
5A④	"	20 C31 + 20 Clay	$119.\overline{5}$	$4.\overline{0}$	$.96\overline{7}$	$206.\overline{2}$	$11.\overline{7}$	.943	

① Absorbance x 10^3 ② mg/l ③ Final values at 25 minutes
④ Initial solids include clay flocculant

System 4

An inorganic metal cation and an anionic polyelectrolyte were evaluated on the storm flow of 8-16-69. As determined by jar tests, optimum concentrations were: 30 mg/l ferric iron (as $FeCl_3$) and 0.5 mg/l PURIFLOC A23. This system did not produce the most rapidly settling floc but was significantly better than the control. The concentration profiles were well spread over the time intervals of sampling. This was contrasted to the rapid sedimentation experienced in the ternary system described previously in which the profiles were closely spaced at the early time intervals.

System 5

A dual system of a clay and a cationic polyelectrolyte was evaluated on the storm flow of 11-18-69. Optimum concentrations as determined by jar tests were: 20 mg/l PURIFLOC C31 + 20 mg/l BAROID HECTORITE clay. The order of addition used in the long-tube test was cationic polyelectrolyte followed by clay. The reverse order was also effective.

The linear correlation of optical and gravimetric solids for the sample used in the evaluation of System 5A is presented in Figure 34. Similar correlations were found for the other samples. The contour maps and reduced concentration profiles described below were developed using optical solids. Gravimetric solids can also be used as a criterion but are more subject to experimental variation.

The contours of optical solids concentration as functions of depth from the bottom and sampling times for System 5A are presented in Figure 35. Concentration decreases with increased settling time and increased depth from the bottom. The contours are generally linear except for those of low concentration where slight variations are magnified.

The optical solids concentrations for the same System 5A are plotted as functions of the logarithms of depth from the surface and sampling time in Figure 36. This system follows Class I sedimentation since the majority of the contours have unit slopes.

The reduced concentration profile of System 5A is presented in Figure 37. The overflow rate is calculated by dividing the depth from the surface by the time of sampling. The establishment of a single curve is another technique to verify Class I sedimentation.

Figure 35

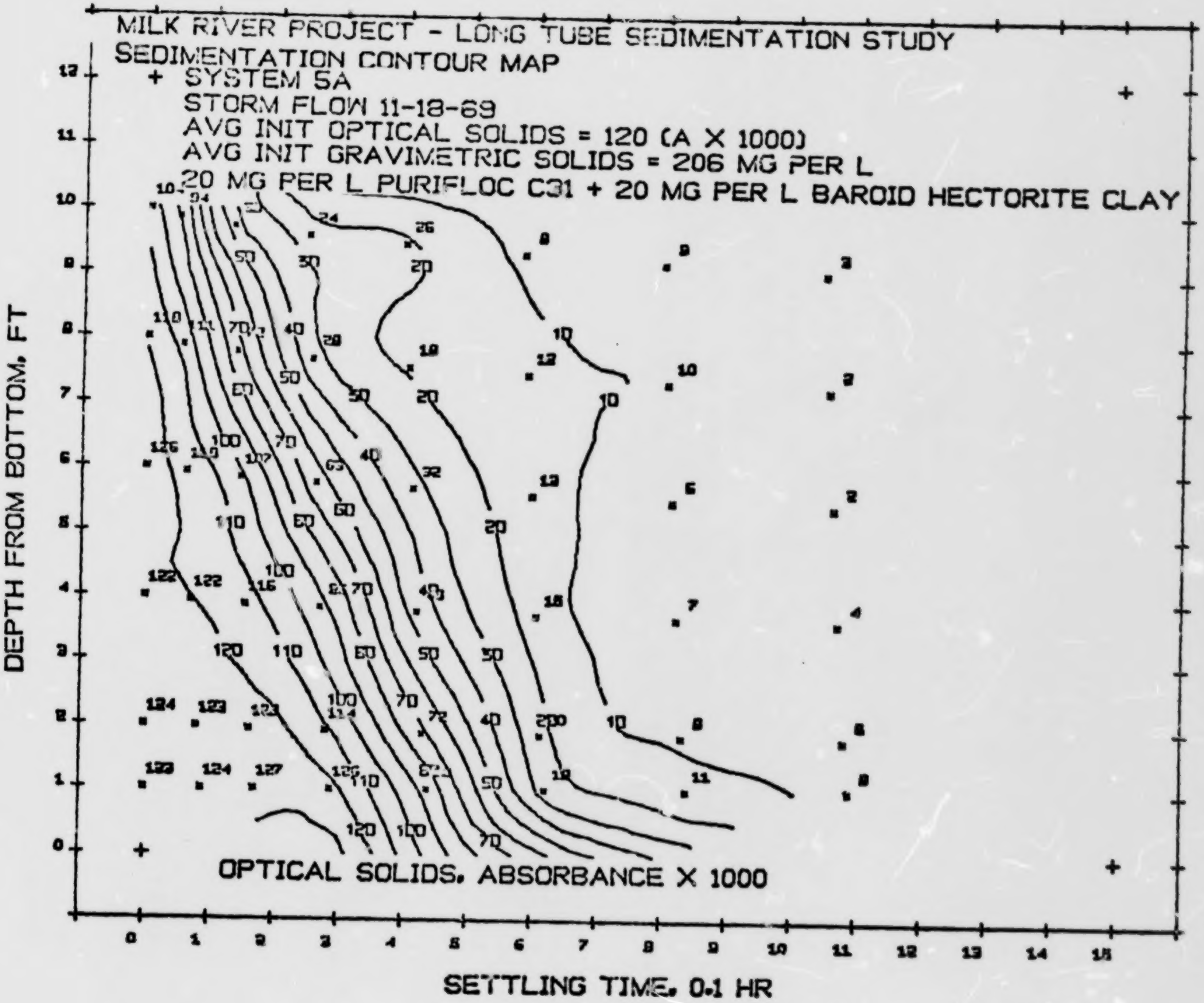

MILK RIVER PROJECT - LONG TUBE SEDIMENTATION STUDY
SEDIMENTATION CONTOUR MAP
+ SYSTEM 5A
STORM FLOW 11-18-69
AVG INIT OPTICAL SOLIDS = 120 (A X 1000)
AVG INIT GRAVIMETRIC SOLIDS = 206 MG PER L
20 MG PER L PURIFLOC C31 + 20 MG PER L BAROID HECTORITE CLAY
OPTICAL SOLIDS, ABSORBANCE X 1000
DEPTH FROM BOTTOM, FT
0
1
2
3
4
5
6
7
8
9
10
11
12
SETTLING TIME, 0.1 HR
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15

Figure 36

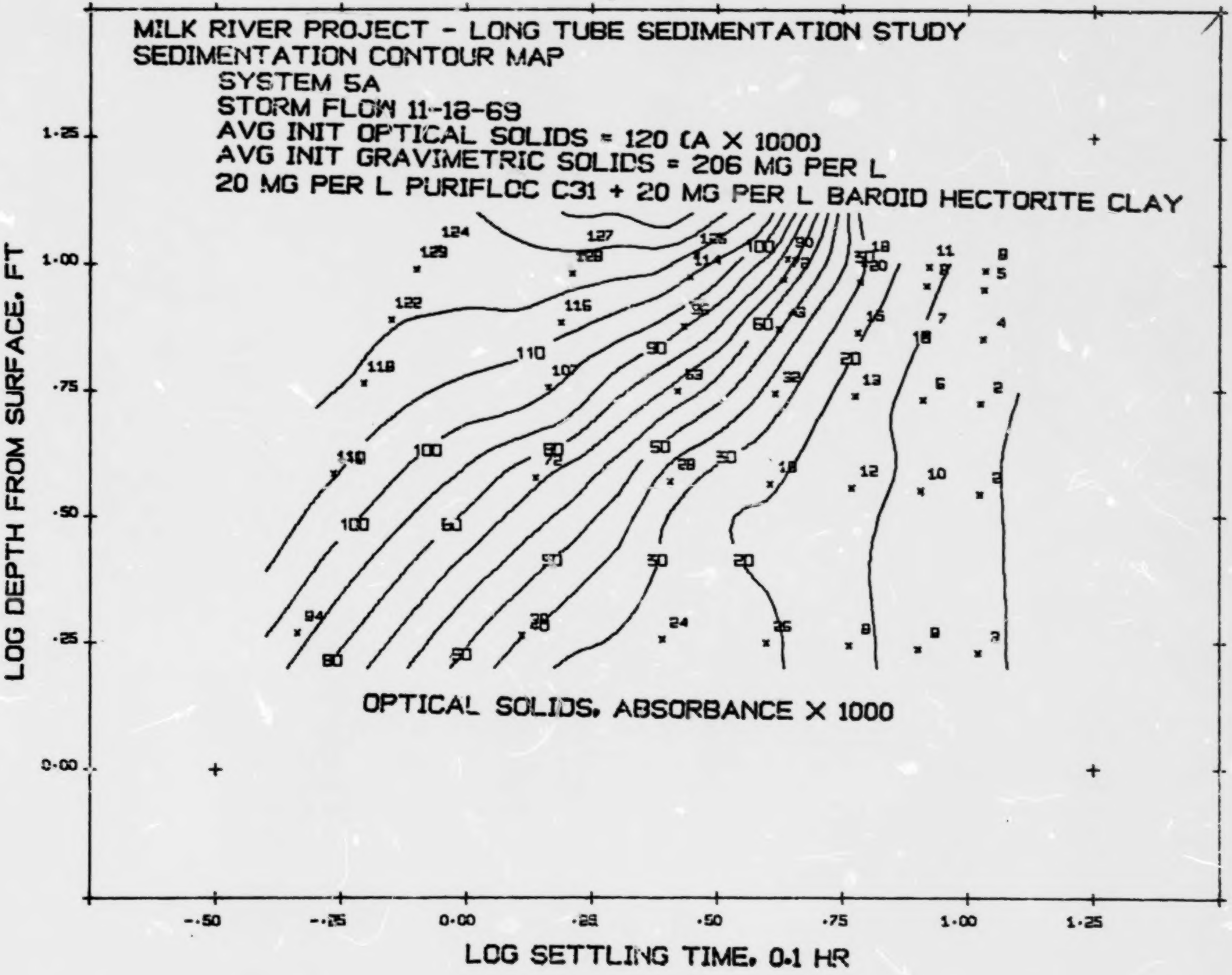
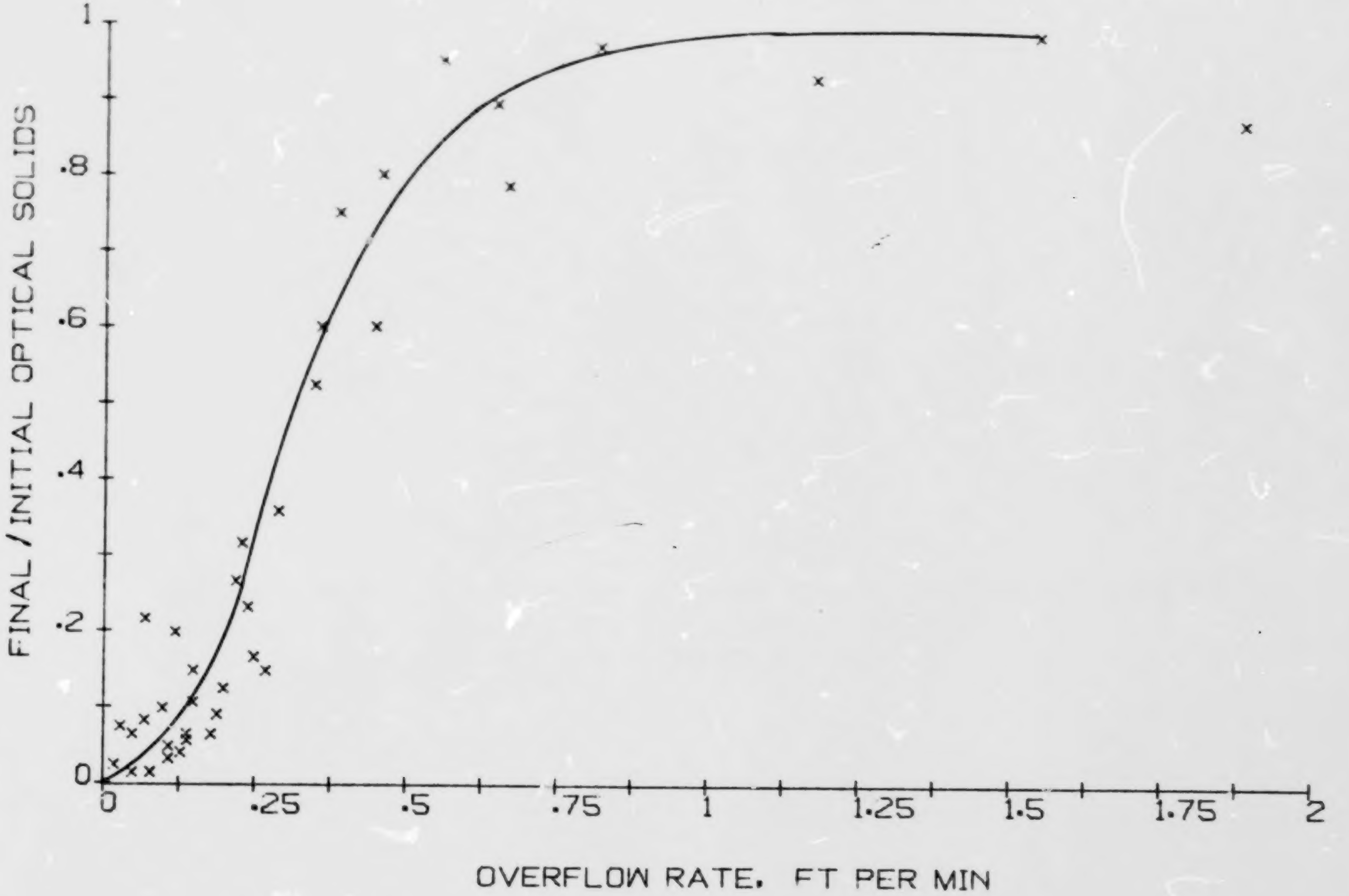

MILK RIVER PROJECT - LONG TUBE SEDIMENTATION STUDY
SEDIMENTATION CONTOUR MAP
SYSTEM 5A
STORM FLOW 11-18-69
AVG INIT OPTICAL SOLIDS = 120 (A X 1000)
AVG INIT GRAVIMETRIC SOLIDS = 206 MG PER L
20 MG PER L PURIFLOC C31 + 20 MG PER L BAROID HECTORITE CLAY
OPTICAL SOLIDS, ABSORBANCE X 1000
LOG DEPTH FROM SURFACE, FT
1.25
1.00
.75
.50
.25
0.00
LOG SETTLING TIME, 0.1 HR
-.50
-.25
0.00
.25
.50
.75
1.00
1.25
124
129
127
128
125
100
90
18
11
8
122
116
114
5
110
90
60
4
7
118
107
63
20
13
6
2
110
100
80
72
50
28
30
18
12
10
2
100
60
50
30
20
94
26
24
26
8
9
3
80
50

Figure 37

MILK RIVER PROJECT - LONG TUBE SEDIMENTATION STUDY
REDUCED CONCENTRATION PROFILE

SYSTEM 5A
STORM FLOW 11-18-69
AVG INIT OPTICAL SOLIDS = 120(A X 1000)
AVG INIT GRAVIMETRIC SOLIDS = 206 MG PER L
20 MG PER L PURIFLOC C31 + 20 MG PER L BAROID HECTORITE CLAY



The fractional loss of optical solids as a function of overflow rate for System 5 is shown in Figure 38. Fractional loss values correspond to the cumulative area under the reduced concentration profile of Figure 37. At an overflow rate of 0.67 fpm one-half of the initial solids are retained and one-half are lost.

All Systems

Values of final/initial optical solids for all five flocculant systems are given in Table XXXIII for selected overflow rates as estimated from the reduced concentration profiles. The clay flocculant amounted to 15-25 percent of the initial solids present for the three systems marked (*). In the strictest sense removal should be based only on the initial sewage solids and be independent of any additional solid flocculants. It was not feasible in the current study, however, to attempt any fractionation of sewage and flocculant solids. The tabulated values for these systems therefore also include removal of the solid clay flocculant.

TABLE XXXIII

VALUES OF FINAL/INITIAL OPTICAL SOLIDS
EXTRAPOLATED FROM REDUCED CONCENTRATION
PROFILES AT SELECTED OVERFLOW RATES FOR
FIVE FLOCCULANT SYSTEMS

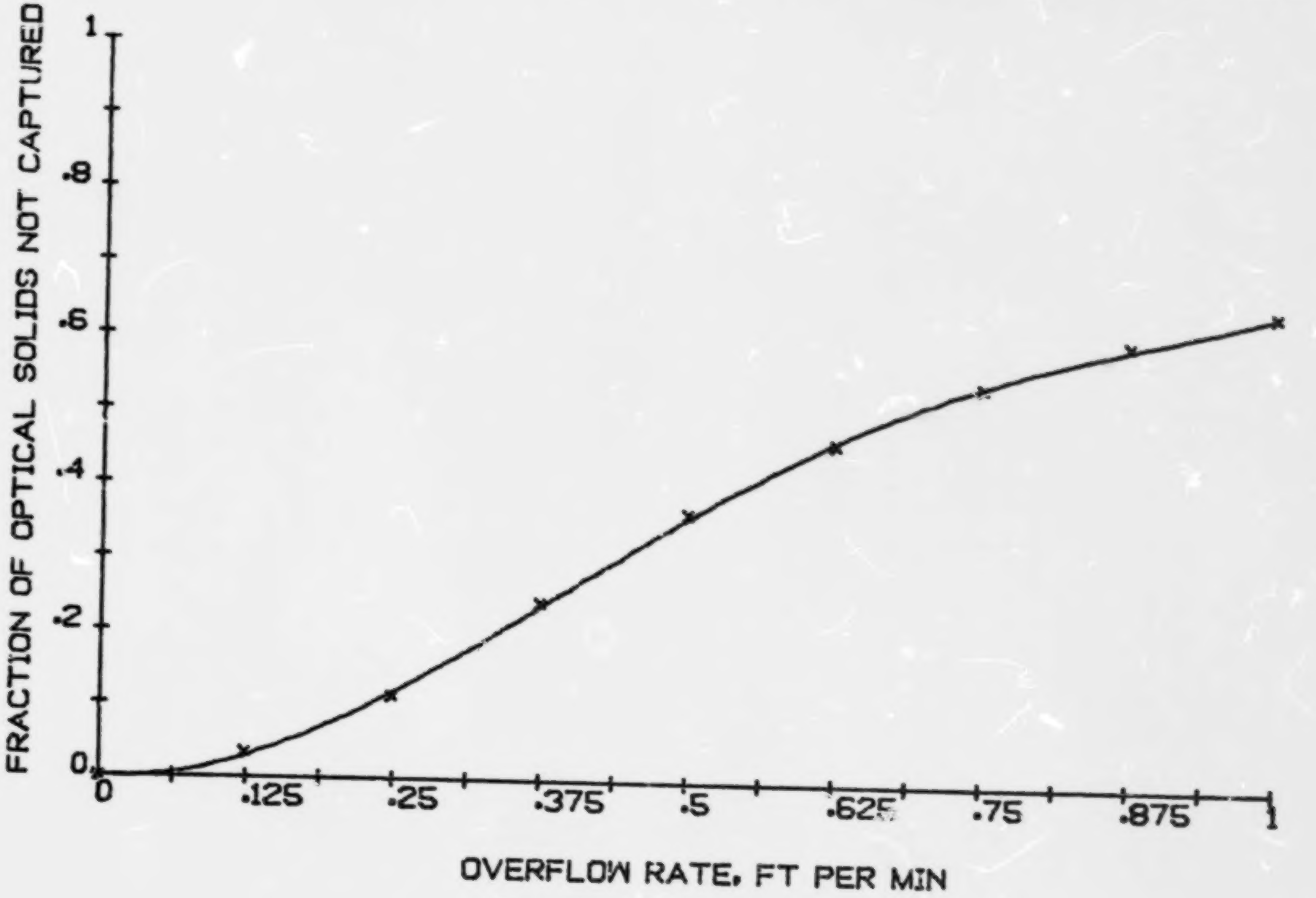
	Final/Initial Optical Solids (%) at Indicated Overflow Rates (fpm)			
System	.25	.5	.75	1.0
1A*	5	16	30	49
1B*	9	35	75	91
2A	62	68	73	80
2B	27	33	41	54
3A	5	13	27	52
3B	20	32	47	67
4A	47	86	95	98
5A*	33	80	95	96

*Initial solids include clay flocculant.

Figure 38

MILK RIVER PROJECT - LONG TUBE SEDIMENTATION STUDY
FRACTIONAL LOSS CURVE

SYSTEM 5A
STORM FLOW 11-18-69
AVG INIT OPTICAL SOLIDS = 120 (A X 1000)
AVG INIT GRAVIMETRIC SOLIDS = 206 MG PER L
20 MG PER L PURIFLOC C31 + 20 MG PER L BAROID HECTORITE CLAY



Fractional losses of optical solids (that is, not captured by sedimentation) at selected overflow rates for all five flocculant systems are given in Table XXXIV. The fraction of solids lost increases as expected as the overflow rate increases for all systems. Although the experimental results are limited, certain observations are possible.

TABLE XXXIV

FRACTIONAL LOSSES OF OPTICAL SOLIDS AT SELECTED OVERFLOW RATES FOR FIVE FLOCCULANT SYSTEMS

	Fractional Losses of Optical Solids (%) at Indicated Overflow Rates (fpm)			
System	.25	.5	.75	1.0
1A	3	7	12	19
1B	4	12	27	41
2A	59	62	65	68
2B	26	28	31	35
3A	4	6	11	18
3B	17	21	27	34
4A	23	47	62	71
5A	11	36	54	65

The flocculant systems giving the best overall performance based on the fractional loss criterion are systems 1A and 3A. Fractional losses for these systems were the lowest of the five systems studied. The flocculated materials formed by Systems 3B, 4A, and 5A were more easily lost at the higher overflow rates. The same flocculants were used in Systems 1A and 1B. The medium flocculated in System 1B, however, was dry-weather flow and may reflect fundamental differences in solids composition. The results of Systems 2A and 2B are questionable because of non-zero intercepts at zero overflow rate for their reduced concentration profiles. In the region of low overflow rates (0-.25 fpm) small variations in measurements of optical and particularly gravimetric solids are magnified. The corresponding samples were collected at extended times when the majority of the solids had already settled.

SUMMARY

The objective of this phase of the Milk River Contract was to obtain sedimentation rate data for combined sewer overflows treated with specified chemical systems. The sedimentation rate was then to be correlated with detention time data derived from the hydraulic model studies. Removal of suspended solids was then to be predicted for specified chemical and hydraulic conditions.

The initial requirement for these studies was the development of a workable Long-Tube Sedimentation Device (LTSD) for evaluation of the sedimentation process. Such a device of sufficient dimensions to approximate full-scale conditions and minimize wall effects and sample volume was constructed. The device consisted of a long tube of 1/4 in. clear plastic, 5" ID x 12', having a capacity of 50 liters. Paddles were spaced 4 in. apart along a central shaft.

Flocculation-sedimentation studies were conducted on samples of combined sewer overflow to evaluate the engineering aspects of flocculant dispersion, mixing, and sample collection during sedimentation. Flocculation was accomplished most uniformly in a holding tank before transfer to the long tube. Dynamic sedimentation at 5 rpm minimized deposition of sedimenting solids on the paddles. Such deposition occurred at 0 rpm and results were skewed.

The feasibility of measuring concentrations of solids by an optical technique was evaluated as a supplement to the conventional gravimetric technique. The correlation between optical and gravimetric solids was linear in the ranges studied. The optical technique was found to be much faster and more reliable than the gravimetric technique.

A program of specified sampling times, depths, and volumes, and procedures to implement such a program were developed. A grid of six sampling depths (1, 2, 4, 6, 8, and 10 ft.) and eight sampling times (0, 4, 9, 16, 25, 36, 49, and 64 min.) was adequate for definition of a given system. The duration of manual sampling by syringe was the limiting factor.

Long-tube sedimentation studies were conducted on samples of both flocculated and unflocculated combined sewer overflows. Sufficient volumes of overflows were collected during representative storm periods. Preliminary jar tests were conducted to determine optimum chemical levels for each selected system.

The sedimentation behaviors of unflocculated combined sewer overflows were evaluated as controls for comparison with chemically-treated overflows. The sedimentation behaviors of combined sewer overflows were evaluated using the most effective flocculant systems as determined from the Flocculant Studies.

The sedimentation data were then correlated with hydraulic data. This involved a data processing program and a graphical presentation program. The regime of sedimentation (Class I or II) was determined by developing contour maps of suspended solids removal as a function of sedimentation time and depth. Discrete sedimentation (Class I) of fully-flocculated solids predominated. Since Class I sedimentation predominated, reduced concentration profiles were developed. Fractional loss profiles were then developed by determining the cumulative areas under the reduced concentration profiles.

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SECTION 8

HYDRAULIC MODEL

INTRODUCTION

It was essential to consider some of the full-scale problems associated with hydraulic deficiencies and flow distributions before results of the flocculant jar tests and the long-tube sedimentation tests could be extrapolated to estimate full-scale removal efficiencies. It was not possible to study these variables in the prototype basin within the constraints of the project because of the infrequency, unpredictability, and variability of storm events. A scale hydraulic model of the Milk River Pumping Station (MRPS) was constructed, therefore, to duplicate in model scale any flow condition or pumping configuration possible in the prototype installation.

The model was designed with geometric similitude to a scale of 20:1, fabricated in sections, and assembled in the flocculant feed room at the MRPS. The model hydraulically simulated the operation of the prototype at equal Froude numbers (V^2/gL). The similitude factors used in the model design are given in Table XXXV. Several views of the model are shown in Figures 39 and 40. Pumping and loading characteristics of the storm pumps were simulated by varying the condition of influent flow. Fresh water was introduced to the model from a constant head tank through a header and any combination of up to seven pump columns. These columns were arranged in the horseshoe pattern of the storm pumps of the prototype.

Experiments were performed to improve the settling characteristics by changing detention time, inlet and outlet conditions, energy dissipation in the approach channel, and elimination of short-circuiting.

Sedimentation or flocculating tests in the model were impractical because of anticipated problems in pumping and size control for scale-up. Modifications of the model were directed towards establishing uniform flow across the width of the basin, elimination of short-circuiting and back flow, and dissipation of potential energy by selected placement of weirs and baffles.

The prototype was originally designed to capture floating material and heavy grit. Suspended solids of low density are not effectively captured under normal operating conditions. The present configuration of the MRPS does not provide a regime favorable to sedimentation of lightweight or even flocculated solids.

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TABLE XXXV

SIMILITUDE OF PROTOTYPE AND MODEL

Scale = 1:20

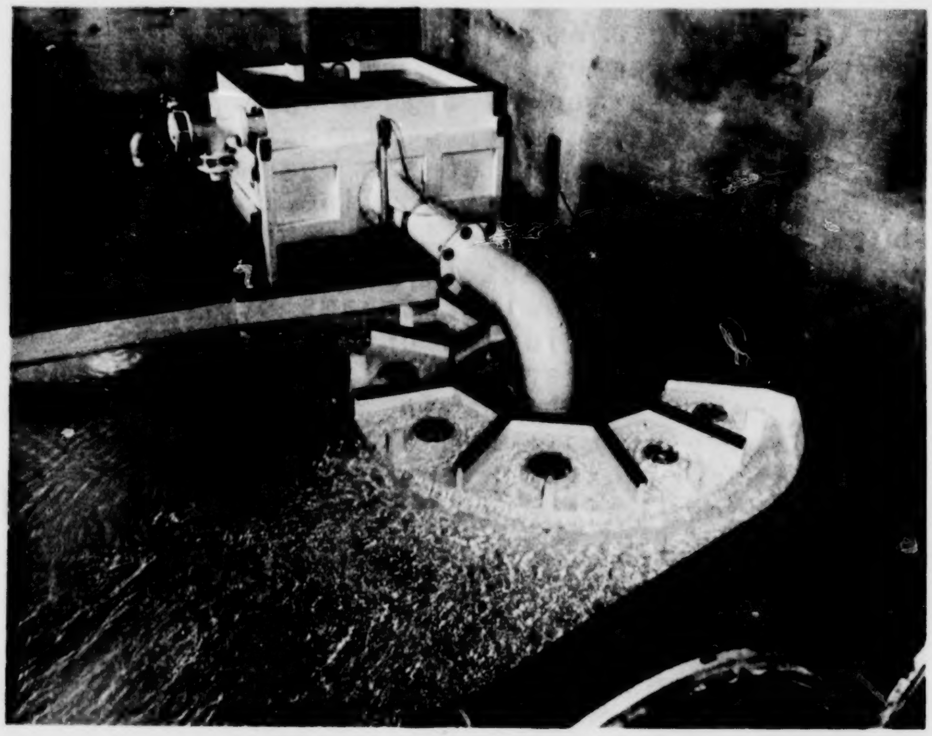
Item	Prototype	Model	
		At Equal Froude No.	At Equal Velocities
Reynolds No.[1]	668,000	7,500	33,600
Velocity (fps)	0.202	0.0456	0.202
Length (ft)	580	29.0	29.0
Width (ft)	150	7.5	7.5
Depth (ft)	10	0.5	0.5
Volume (gal)	3.8×10^6	475	475
Time	1	.224	.0500
Froude No.[2]	$.266 \times 10^{-5}$	$.266 \times 10^{-5}$	5.32×10^{-5}
Flow (gpm)			
(one small pump)	137,000 (305 cfs)	75.4	336
(one large pump)	184,000 (410 cfs)	103	459
(all seven pumps)	1,095,000 (2450 cfs)	611	2,720
Overflow Rate (gpd/sq ft)	2,700	604	2,700

[1] $N_{Re} = \frac{LV\rho}{\mu}$

[2] $N_{Fr} = \frac{V^2}{gL}$

Figure 39

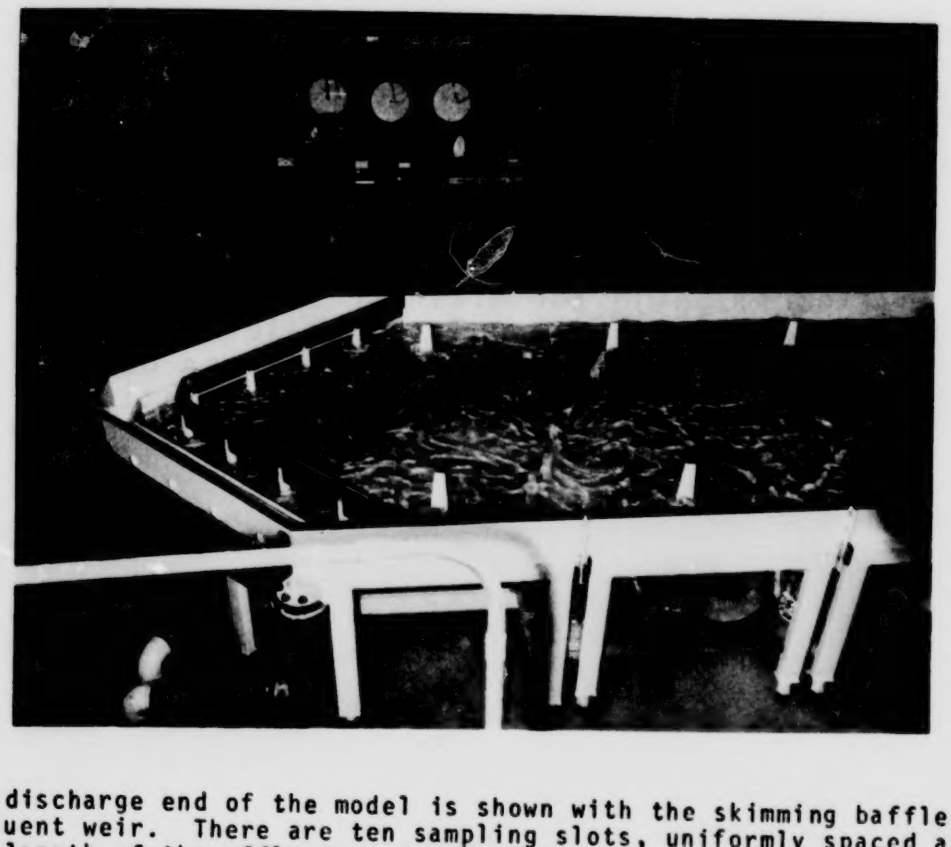
Milk River Pumping Station - Basin Model Inlet



The constant-head tank, the discharge pipe from the tank and a portion of the basin including the influent pump column ports and bays, the distribution plane, terminating at the submerged weir - the white sloping transverse band on the model, and baffles protecting the discharge ports in the basin bottom are shown.

Figure 40

Milk River Pumping Station - Basin Model Discharge



The discharge end of the model is shown with the skimming baffle and effluent weir. There are ten sampling slots, uniformly spaced along the length of the effluent weir numbered A-0 to J-9, starting at the left (lower center).

ANALYSIS OF EXISTING BASIN

Effective removal of appreciable amounts of lightweight suspended solids cannot be expected in operating the existing MRPS as noted above. Sedimentation of lightweight suspended solids requires relatively gentle, well distributed, steady-state input of flow. The arrangement of the pumps coupled with intermittent pumping at high rates combine to prevent transverse distribution of the influent flow. Short-circuiting occurs almost immediately and persists throughout the length of the basin, resulting in fast flowing streams overflowing the effluent weir at various locations along its length. Dead spots and areas of backflow have the effect of reducing the active sedimentation area of the basin. The dead spots are caused by diagonally-impinging short-circuiting streams and reversed flows which are caused by the angular horizontal configuration of the effluent weir and skimming baffle.

The nominal detention time and overflow rates during operation of one small, one large, or all four small and three large pumps at the MRPS are shown in Table XXXVI.

TABLE XXXVI

NOMINAL DETENTIONS AND OVERFLOW RATES
IN MILK RIVER RETENTION BASIN

Flow		Nominal Detention	Overflow Rate
(cfs)	(gpm)	(min)	(fph)
305	137,000	27.8	18.5
410	184,000	20.7	24.8
2450	1,100,000	3.5	148.0

Nominal detention is found by dividing the capacity of the basin in cubic feet by the input in cubic feet per minute. The nominal detention time available for sedimentation by plain settling (no flocculation) or for flocculated solids is limited, even if more favorable conditions of flow existed.

Overflow rates (volume input in cubic feet per hour divided by basin surface area in square feet) of from 18.5 to 148 fph are also significantly high for plain sedimentation or Class I sedimentation (fully flocculated solids) to occur. As seen earlier, simultaneous operation of all seven pumps occurs only very infrequently. Rectangular sedimentation basins are generally designed to a length to width ratio (L/W) of not less than about 5:1 to reduce the possibility of short-circuiting. The L/W ratio of the basin at the MRPS is about 2.5:1.

The active depth of the basin is about nine feet and is satisfactory for sedimentation. Depth is reduced, however, to about 3.2 feet for a distance of 75 feet as the influent flows over the distribution plane prior to entering the basin proper. The rate of progression down the distribution plane is 38 fpm (one small pump operating) and is sufficiently low to permit sedimentation of some grit and possibly some lightweight solids. Since the rate of progression in transverse distribution is not uniform the effectiveness of the area over the distribution plane is unpredictable.

IMPROVEMENTS TO THE MILK RIVER PUMPING STATION

A number of modifications to improve the capability of the MPRS were suggested based on studies of plant records, current operating conditions, and dye tests in the model. Continuous (staged) pumping would considerably reduce average pumping rates. Placement of transverse weirs would essentially prevent short-circuiting and reduce the possible adverse effects of density currents. Both modifications would increase the efficiency of the basin to handle solids removal for specified flows and suspended solids loadings either in plain settling or after chemical treatment.

Combined sewer overflows amount to about 5.5 million cubic feet (41.1 million gallons), 90 percent of the time. The storm sewers can contain five million gallons and the wet well about 0.3 million gallons. There are four small and three large stormwater pumps at the MRPS rated at 305 and 410 cfs (137,000 and 184,000 gpm), respectively, at pumping heads of 33 feet. Continuous pumping of combined sewer overflow at approximately one-third the capacity of one small pump is feasible and desirable when possible, especially in sedimentation of chemically treated suspended solids. Reduced pumping rates (102 cfs, 45,800 gpm) can be accomplished by replacement of one of the small pumps with one allowing a variable discharge rate, or by adding a drain for recycling about two-thirds (203 cfs, 91,100) of the input from one small pump to the wet well without impairing the ability to deliver combined sewage at the rated capacity of all seven pumps.

Satisfactory transverse flow distribution of influent cannot be accomplished at the MRPS by reducing pumping rates or by tandem pumping within the existing design. The addition of baffles, weirs, or other flow distributing devices is essential to provide rapid transverse distribution of basin influent.

Gross short-circuiting and reverse flowing streams could be visually observed in the prototype and in the unbaffled model. The short-circuiting, dead spots, and reverse flows in the unbaffled model when operating one small corner pump at a reduced flow, equivalent to about 82 cfs (36,800 gpm) input to the prototype is shown in Figure 45. Much the same pattern of uncontrolled distribution occurs in the model (or prototype) at higher rates of flow.

Adverse effects are lessened when operating a more centrally-located pump. No pump is located on the long axis of the prototype basin, however, and storm flow is skewed under any condition of pumping. Operation of two opposing pumps provides input excessive for good sedimentation under existing conditions. Uncontrolled flow restricts prediction of solids removal even though the detention times may compare favorably with those found when weirs are used.

QUALITATIVE DYE STUDIES

Qualitative tests of the dispersion of Bismark brown dye were conducted for a range of influent flows on the open model prior to testing the placement of weirs and baffles. The need for modifications for obtaining improved detention efficiencies in the model were emphasized by the results of these tests. Detention efficiencies could be improved by reducing short-circuiting and hydraulic heads, and by preventing back flow caused especially by the configuration of the skimming baffles.

These objectives were approached by optimizing arrangements of baffles and/or weirs in the model. Two weirs, each with 15 equally-spaced 90° notches, were fabricated and tested in the model. The need for more and varied weirs and baffles was stressed by the results of these preliminary tests. Several additional weirs and baffles were fabricated from aluminum stock.

The 90° notches in the original weirs were also satisfactory in the design of new weirs to be placed on the distribution plane and in the vicinity of the upstream end of the basin proper (weirs A, B, E). Notches in A and B weirs were made somewhat deeper than the original weirs. This reduced the water level at inactive pump columns when these weirs were used on the distribution plane. A and B weirs were similar and made in pairs with staggered notches. The overall heights of the B weirs were somewhat less than those of the A weirs. Weir E was similar to A and B but was higher than an A weir for effect in basin placement. Weir G was designed with fifteen 55° notches for use in the general vicinity of the middle of the basin. Weir H was designed with twenty-two 40° notches. The notches in G and H were cut deep to avoid converting kinetic energy into potential energy.

Qualitative and quantitative dye tests were performed using the improved weirs and baffles in various combinations and locations. The identity and location of the weirs and baffles in model tests are shown in a plan outline of the model basin in Figure 41. Notations of operating conditions and comments are recorded. Selected data from the sketches are tabulated in Table XXXVII. Most tests were performed with only one small corner pump (#7) in operation. Flow from this location was considered most difficult to distribute transversely. Conversion factors between prototype and model are tabulated in Table XXXV.

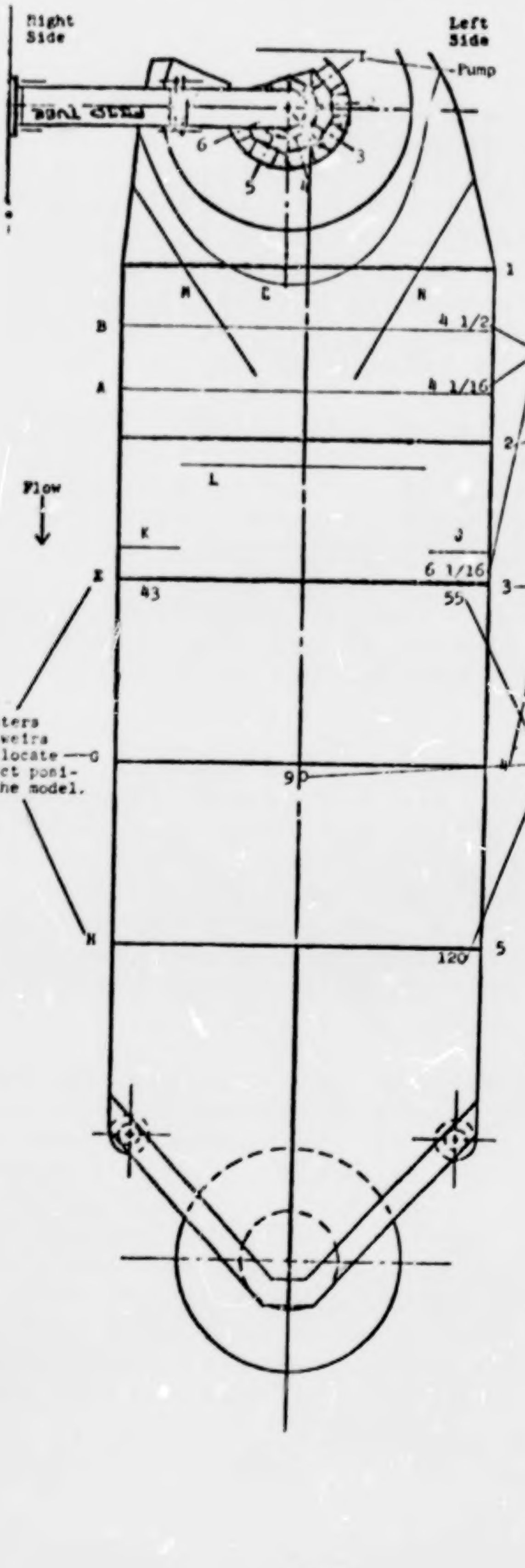
Tests performed with baffles placed on the distribution plane or in the basin were unsuccessful in preventing short-circuiting or in substantially consuming the hydraulic head. The inclusion of a curved weir circumscribing the pump discharge bay provided some limited flow distribution and consumption of hydraulic head. The weir used in these tests was evidently too low to be effective and its use was discontinued in favor of pairs of weirs placed across the distribution plane. In most subsequent tests combinations of A and/or B weirs were used.

In some runs a chain curtain was also located on the distribution plane or in the basin. It did not appear to contribute to transverse distribution of the dye front nor to appreciable consumption of hydraulic head. Water overflowed the inactive pump ports with either one small pump or one large pump in operation due to height of the A weir in runs where two A weirs or one B and one A weir were used. Therefore attention was directed to evaluation of performance when B weirs were tested.

Placement of weirs in the basin was important in reducing velocity head and in developing transverse distribution of dye. Conditions improved as the number of weirs was increased. Three weirs in the basin seemed necessary when operating one small pump to effectively complete transverse distribution of the dye before it reached the effluent weir. The weir farthest downstream was most effective in providing uniform flow to the overflow weir when it had many (22) 40° V-notches. Such a weir (H) was also very effective in preventing backflow of water caused by the V-shaped configuration of the effluent weir. It was also found desirable

Figure 41

SCHEMATIC OF MILK RIVER PUMPING STATION MODEL


Right Side
Left Side
Pump
6
5
4
3
1
M
C
N
B
A
4 1/2
4 1/16
2
L
K
J
E
6 1/16
43
55
3
G
90
4
H
120
5
Flow

The flow rates for the two pumps used were:

Pump 4: 103 gpm
Pump 7: 75.4 gpm.

If two V-notch weirs (W) were used between sections 1 and 2 then the V-notches were staggered.

These numbers refer to the depth of the water in inches at that location.

This locates the bottom of the submerged weir.

These arbitrary numbers locate the joints on the model.

These letters refer to weirs used and locate their exact position in the model.

These numbers refer to the time in seconds when the dye arrived at that location.

TABLE XXXVII

FORMAL DYE TEST	RUN	PUMP	GPM	WEIRS A-H BAFFLES K-N							HEAD DETENTION		
				LOCATION							TIME, SEC		
					1		2	3	4	5	L	C	R
	1	7	75	C	A,22			G		H	160		145
	2	7	75	C	B,10	B 22		G		H	180		180
	3	7	75	C				G		H	180		120
	4	7	75	C	K M N			G		H	150		150
	5	7	75		M N		L	G		H	105		150
	6	7	75		A,7	A 21 1/8		G		H	180		180
	7	7	75		A,7	A 21 1/8		G	K,J	H	170		160
	8	7	75		A,7	A 21 1/8	K J	G		H	160		190
	9	7	75		A 11 3/8	A 21 1/8		G		H	155		185
	10	7	75		B 11 1/8	A 21 1/8		G		H	230		210
	11	4	103		B 11 1/8	A 21 1/8		G		H	125	125	125
	12	4,7	178		B 11 1/8	A 21 1/8		G		H	90	130	-
	13	7	75		B 11 1/8	A 21 1/8	F	G		H	205	205	205
	14	7	75		B 11 1/8	A 21 1/8	F	G		H	120	195	170
1	15	7	75		B 11 1/8	A 21 1/8	F	G		H	150	150	150
2	16	4	103		B 11 1/8	A 21 1/8	F	G		H	120	120	120
	17	7	75		B 11 1/8	A 21 1/8	F	H			210		190
	18	7	75		B 11 1/8	A 21 1/8	F	H		G		180	
	19	7	75		B 11 1/8	A 21 1/8		E	G	H	140		140
	20	7	75		B 10 3/4			E	G	H	130	195	140
	21	7	75		B 10 3/4		F	E	G	H	135		135
	22	7	75		B 10 3/4	B 20 3/4		E	G	H	165		165
3	23	7	75		B 10 3/4	B 20 3/4		E	G	H	138		138
	24	7	75		B 11 3/4	B 21 3/4		E	G	H	170		150
	25	7	75		B 11 3/4	B 21 3/4		E	G	H	165		165
4	26	4	103		B 11 3/4	B 21 3/4		E	G	H	86		86
	27	4	103		B 11 3/4	B 21 3/4		L	K J	H	100		100
	28	4	103		B 11 3/4	B 21 3/4	A	E	G	H	120	175	120
5	29	4	103		B 11 3/4	B 21 3/4	A	E	G	H	89		89
	30	7	75		B 11 3/4	B 21 3/4	A	E	G	H	170	200	200
6	31	7	75		B 11 3/4	B 21 3/4	E		G	H	141		141
	32	7	75		B 11 3/4	B 21 3/4	E		G	H	170	240	170
7	33	4	103		B 11 3/4	B 21 3/4	E		G	H	118		118

Table title: VISUAL AND FORMAL DYE TESTS [illegible] VARIOUS FLOWS, BAFFLE AND WEIR PLACEMENTS. JUNE 18-20, 1969.

TABLE XXXVII (Cont.)

VISUAL AND FORMAL DYE TESTS AT VARIOUS FLOWS,
BAFFLE AND WEIR PLACEMENTS NOV. 11-13, 1969, DEC. 12, 1969.

FORMAL DYE TEST	RUN	PUMP	GPM	WEIRS A-H LOCATION						HEAD DETENTION TIME, SEC		
				1		2	3	4	5	L	C	R
	34	1	28.6	B 11 3/4	B 21 3/4	E		G	H	390		390
11	35	1	28.6	B 11 3/4	B 21 3/4	E		G	H	415		415
12	36	1	23.5	B 11 3/4	B 21 3/4	G	18 3/4		H	465	420	420
	36A	1	23.5	B 11 3/4	B 21 3/4				H	465	690	570
13	37	7	103	B 11 3/4	B 21 3/4	E		G	H	135		135
14	38	7	40.8	B 11 3/4	B 21 3/4	E		G	H	255		255
15	39	7	20.5	-	-	-	-	-	-	180		330
16	40	7	40.0	-	-	-	-	-	-	-	-	-
17	41	7	75.0	-	-	-	-	-	-	-	-	-

Run 36 G 18 3/4 in. downstream from 2.

Footnotes:

L = left C = center R = right
Refer to Figure 41 for location of C-weir
Numbers in Location 1 refer to the distance of the weir downstream from 1
Refer to Figure 41 for typical location of baffles K-N
F is located at 2 - the joint between sections 1 & 2 which is the bottom of the submerged weir.
Not all baffles used in combination with all weirs
Location 1 joint between model sections 1 and 2
2 joint between model sections 2 and 3
3 joint between model sections 3 and 4
4 joint between model sections 4 and 5
5 joint between model sections 5 and (6-7).

to place an E-weir close to the end of the submerged weir (upstream end of basin) and another weir (G) with 55° V-notches 4 inches from the division between model sections four and five. This arrangement of weirs in the lower reaches of the basin plus two B weirs on the distribution plane was an effective combination.

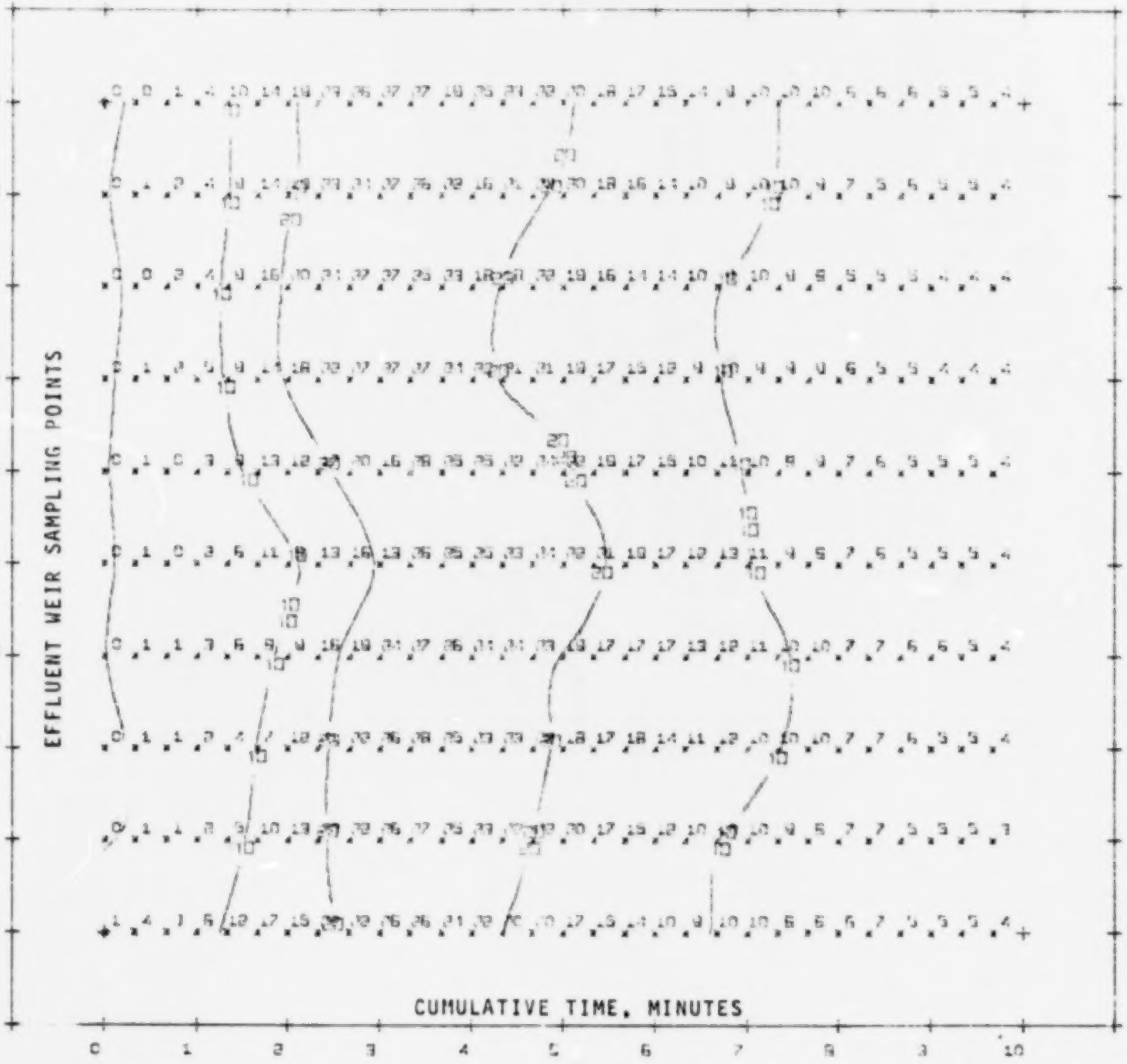
QUANTITATIVE DYE STUDIES

The degrees of effectiveness of the individual weirs and their placement were evaluated in more detail in quantitative tests of the distribution of dye in the model. Contour maps of dye absorbances (proportional to concentration) were prepared for several quantitative studies. Dye concentrations were measured with a probe or fixed unit colorimeter. The abscissae of these plots are the cumulative times (min) after the dye fronts are first observed to overflow the effluent weir. The detention times are the times the dye required to reach the effluent weir. The ordinates are the ten positions across the effluent weir (0 = A, 1 = B, ..., 9 = J). The contoured absorbances (x 10^3) represent successive contours of dye concentration, "frozen" as they crossed the effluent weir and continue undisturbed through an imaginary projection of the model.

The effectiveness of the weir design and placement in the model when one small corner pump (7) was in operation is illustrated in the contour map (Figure 42) of dye concentration from the highly significant Run 31. Run 33 was performed to provide data for operation of one large off-center pump (4). Results obtained for pump 4 were satisfactory even though optimization of flow control for a corner pump was expected to be the most difficult. The lack of good flow distribution was probably due to the location of the pump. Run 37 was therefore made with 103 gpm flowing from pump 7.

Comparison of Run 37 with Run 33 shows some improvement in detention time, 135 seconds versus 118 seconds, when using the corner pump. Both contour patterns were similar and showed irregularities for periods between approximately one and three minutes (Run 37) and four minutes (Run 33). The 103 gpm input to the model would therefore be excessive. Support for this contention is found in other contour maps. Higher weirs on the distribution plane cannot be used to improve this situation, without increasing the height of the (inactive) pump columns to avoid their flooding. Increasing the height of the pump columns is impractical.

Figure 42
MODEL DYE TEST



TEST 6, RUN 31, PUMP 7, FLOW 75 GPM, DETENTION 141 SEC, 6/20/69

There may be good reason, however, to postulate that the increased detention time experienced in Run 37 was due to using a corner pump for this amount of flow, rather than pump 4 (which had been used for all other similar inputs). In which case it may be assumed that the location of the B weirs further downstream on the distribution plane would have been more effective when using pump 4 and 103 gpm input.

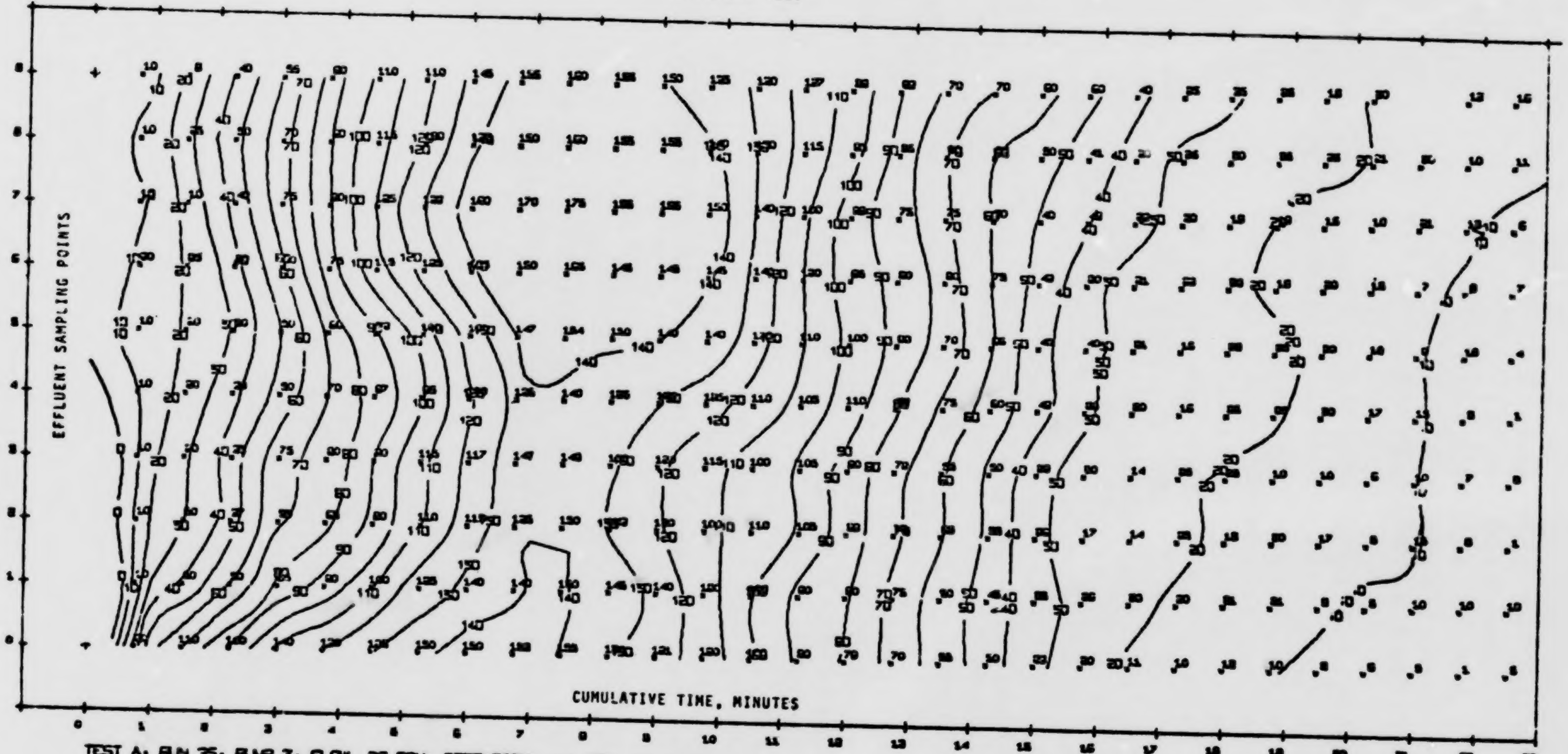
The contour map of dye concentrations from Run 35 (Figure 43) is also highly significant in that it represents model performance for reduced flow of 28 gpm (equivalent to 116 cfs [52,000 gpm] in the prototype) or approximating one-third the capacity of one small pump. Good flow control and a total detention time (elapsed time after adding dye to the center of the pump port until the dye disappears over the effluent weir) of about 28 minutes (equivalent to about 125 minutes in the prototype) is indicated by the contour pattern. Performance in the model for a flow of 40.8 gpm (equivalent to 166 cfs [74,500 gpm] in the prototype) or approximately one-half the capacity of one small pump is shown in the contour map of Run 38 (Figure 44). Good flow control again is indicated for a total detention of about 16 minutes (equivalent to 71 minutes in the prototype). The contour map of Run 39 (Figure 45) represents model performance for a flow of 20.5 gpm (equivalent to 83 cfs [37,200 gpm] in the prototype) with no weirs used in the test. The total detention time, however, is the same in Run 38 where weirs and double the input were tested. Runs 40 to 42 were performed at other flows without weirs in the basin. The total detentions are only indications of the improvements that can be expected in the standard and average detentions determined by computation.

EFFECT OF BAFFLES AT REDUCED FLOW RATES

It has already been noted that intermittent operation of the MRPS leads to inefficient, non-steady state settling conditions in the retention basin. Variable speed pumping at rates below 305 cfs would permit relatively steady-state pumping and a considerable increase in basin detention for up to about 45 percent of the total flow at the MRPS.

Dye studies in the model at the reduced flow rates have shown that baffling can improve retention efficiency in the basin markedly. A summary of the model dye studies showing the increase in basin detention time achieved for plug flow under unbaffled and optimally baffled conditions is contained in Table XXXVIII.

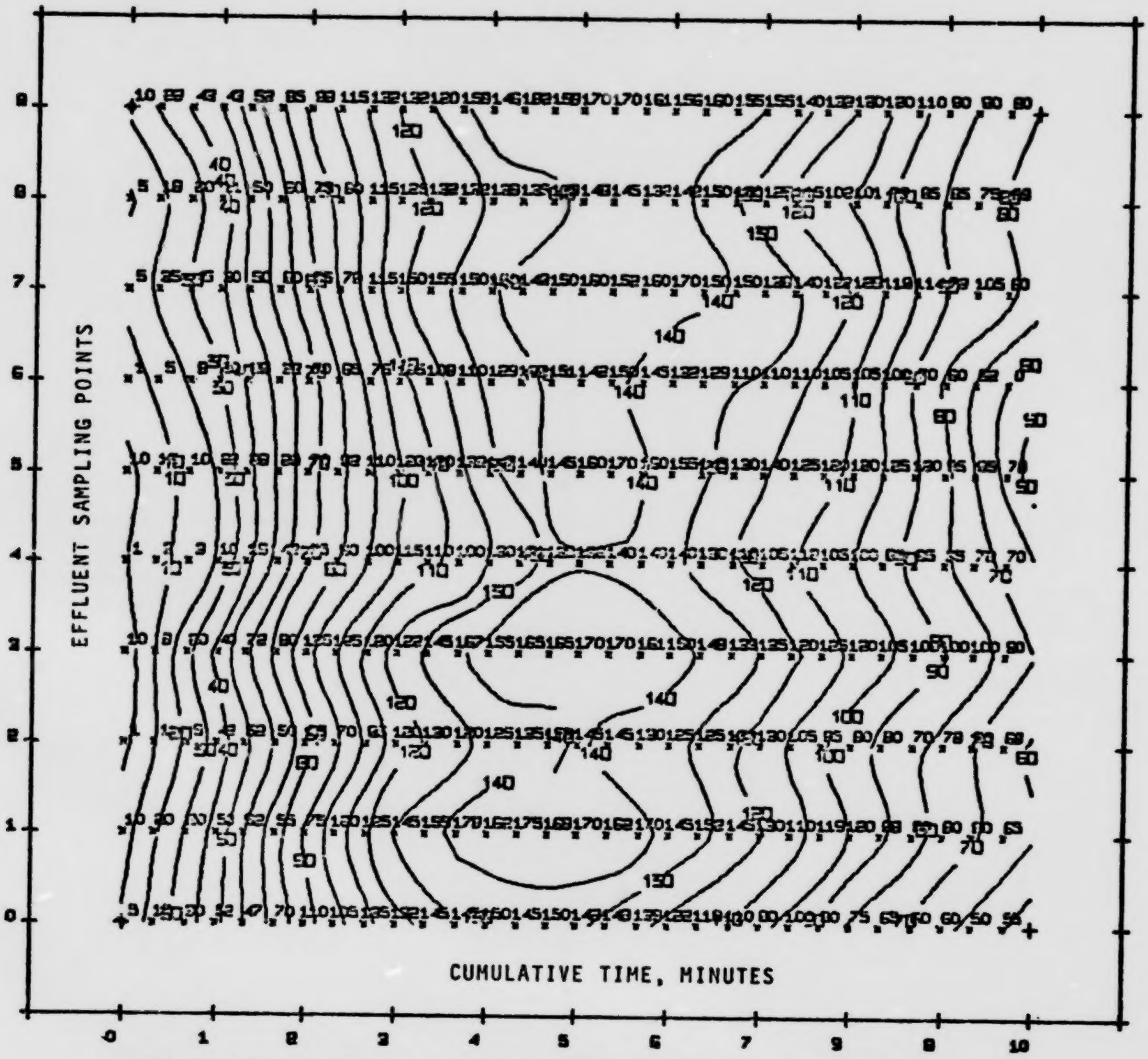
Figure 43
MODEL DYE TEST



TEST A, RUN 35, PUMP 7, FLOW 28 GPM, DETENTION 415 SEC, 11-11-68

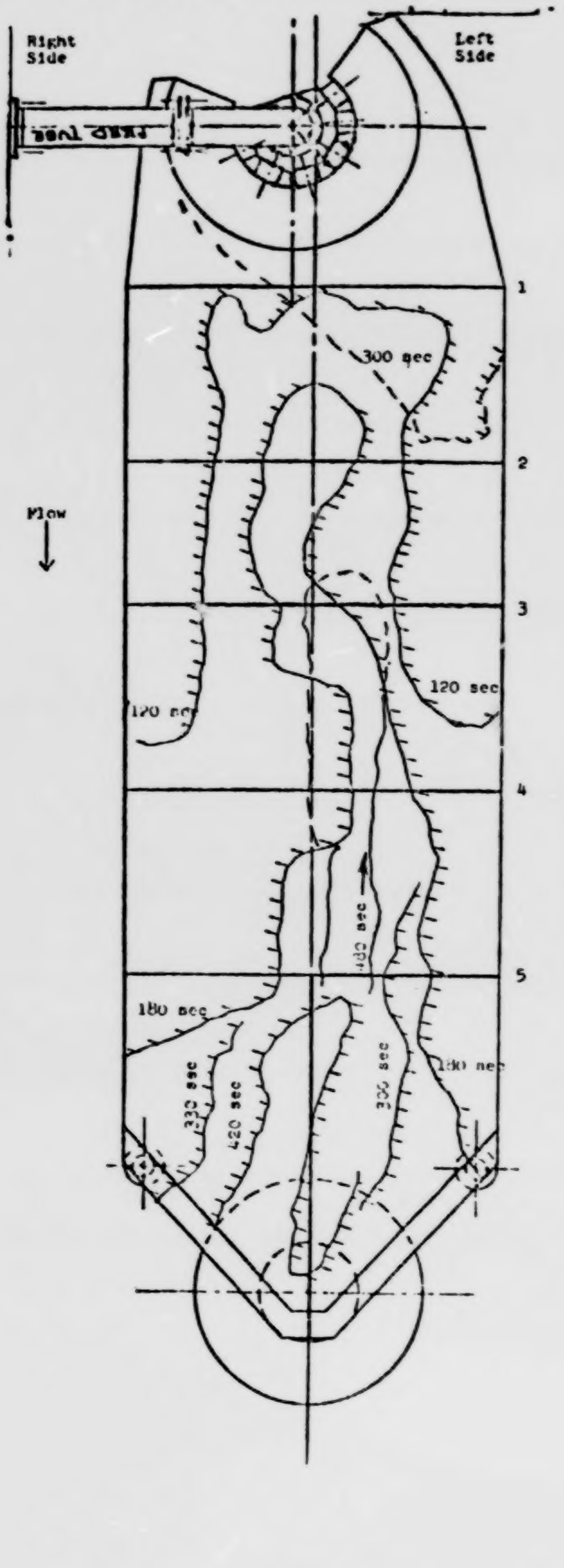
Figure 44

MODEL DYE TEST



TEST C, RUN 38, PUMP 7, FLOW 40.8GPM, DETENTION 255 SEC, 11-13-69

Figure 45
MODEL DYE TEST USING PUMP NUMBER 7 at 20.5 GPM


Right Side
Left Side
Feed Tube
1
2
3
4
5
Flow
300 sec
120 sec
120 sec
480 sec
180 sec
330 sec
420 sec
300 sec
180 sec

Date: 11/13/79

Test No.: 1, Run No.: 39

Pump No.: 7

20.5 gpm

Weirs or Baffles:

None

Dye: Bismark brown

Conc., %: 10

Vol., cc: 100

Water Temperature, °F:

53

Comments:

Water level at inactive pump discharge ports:

Water did not overflow effluent weir at sampling points 8, 9, 10
At 210 sec sampling started
At 540 sec heavy concentration overflowing
At 840 sec color medium to light, patchy
At 1200 sec sampling stopped
At 1260 sec fairly even distribution of light to medium dye from Location 5 to effluent weir

TABLE XXXVIII

EFFECT OF BAFFLES UPON DETENTION TIME AT VARIOUS RATES OF PUMPING

Test No.	Run No.	Date	Model Rate (gpm)	Prototype Equivalent Rate (cfs)	Prototype Equivalent Rate (gpm)	Detention (min) Equiv. Gross	Detention (min) Theoretical
All Baffles (1969)							
A	35	11/11	28	116	52,000	66.1	76
C	38	11/13	41	166	74,500	42.4	52
B	37	11/13	103	410	184,000	20.1	21
No Baffles (1969)							
D	39	11/13	21	83	37,200	30.3	101
F	40	12/12	40	163	73,100	13.4	53
E	41	11/13	75	305	137,000	-	-
G	42	12/12	103	410	184,000	8.9	21

The retention times obtained under the unbaffled and optimally baffled conditions plotted against the theoretical retention in the basin are shown in Figure 46. The increase in retention efficiency from baffling is clear. From a practical standpoint, the optimum baffling configuration would be costly so a compromise between cost, increased efficiency and hydraulic limitations is necessary. The hydraulic problem of passing 25 to 100 percent of the overflow volume through a baffled system under increased rates of flow must also be considered.

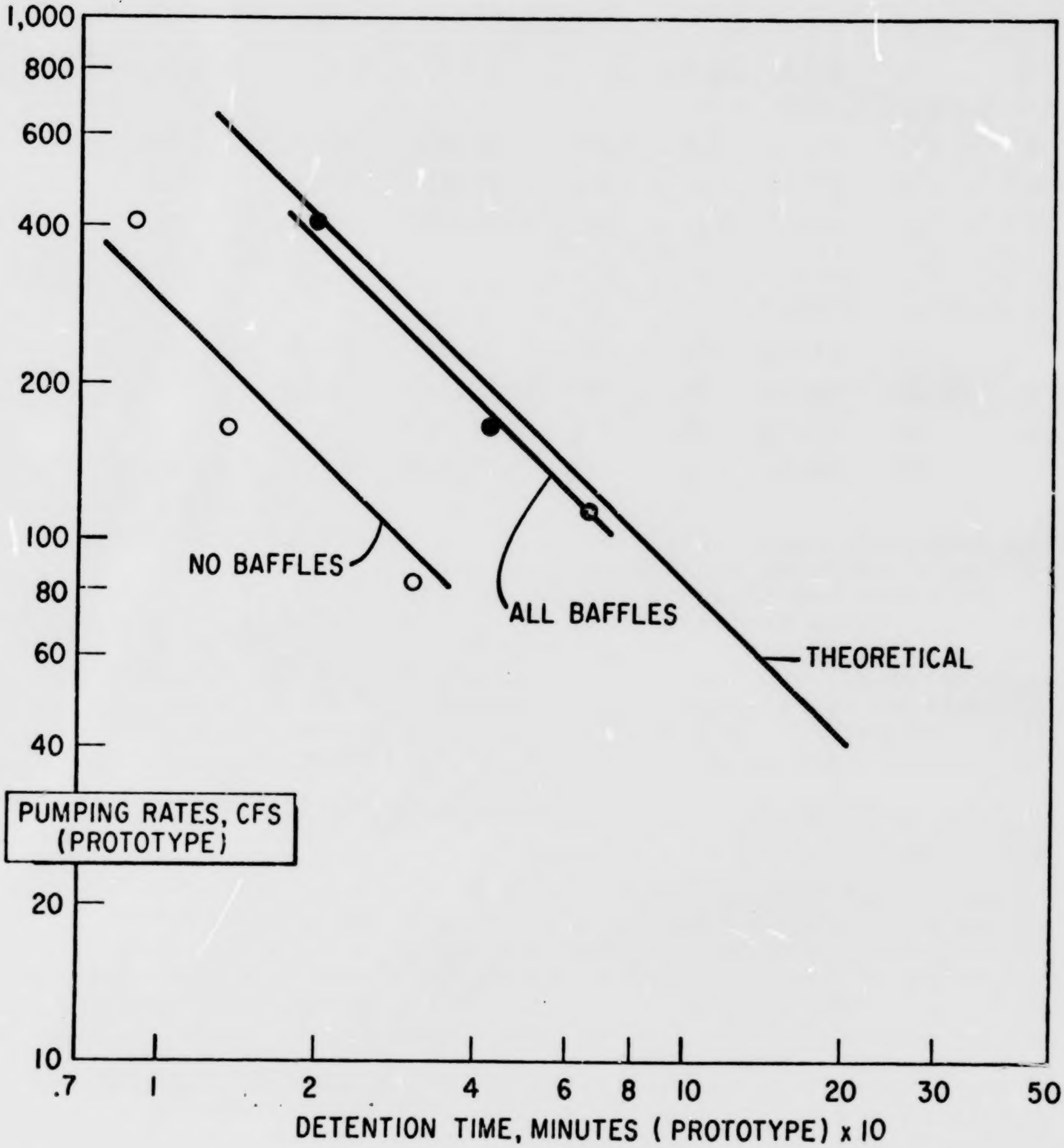
OPTIMUM CORRELATION OF FLOCCULATION, LONG-TUBE SEDIMENTATION AND MODEL TEST RESULTS

Prediction of removals of discrete fully-floccculated (Class I) solids from settling basins can be based on batch settling tests and on model dye tests. The analysis of sedimentation conducted in long vertical tubes is discussed in a previous section of this report. Dye tests in the hydraulic model were analyzed as described below.

A typical dye test was conducted on a basin model by injecting a slug of dye or other marking substance into the basin influent such that it was mixed uniformly into all elements of fluid entering the basin during a very short time interval following time zero. Effluent was sampled periodically at a number of stations preferably uniformly spaced along

Figure 46

EFFECT OF BAFFLES AT REDUCED FLOW RATES



the discharge weir. All effluent samples were analyzed for dye concentrations. The average detention time of dye was determined for each station[2]. The local overflow rate (basin depth divided by average detention time was determined for each station. The predicted local value of the fractional solids not captured, (F_p), was determined. A typical plot of the local values of F_p vs. distance (l) along the weir, is shown in Figure 47. The average value of local F_p is defined as:

$$F_b = \frac{\int_o^W F_p \, dl}{W}$$

This quantity represents the fraction of the initial solids which is predicted to overflow the basin under the conditions of the dye test.

A basin operating at an overflow rate different than that used during the dye test should have approximately similar flow patterns in a kinematic sense. In such a case, all local average detentions will change by a constant factor, which is equal to (test overflow rate)/(new overflow rate). Under the assumption that this is true, predictions of solids loss at overflow rates can be made by multiplying the observed values of detention times by this overflow factor, and then carrying the calculations through as before.

Correlation of results from flocculation and sedimentation in long-tube tests and the dye tests are developed sequentially using the procedure described earlier in this report.

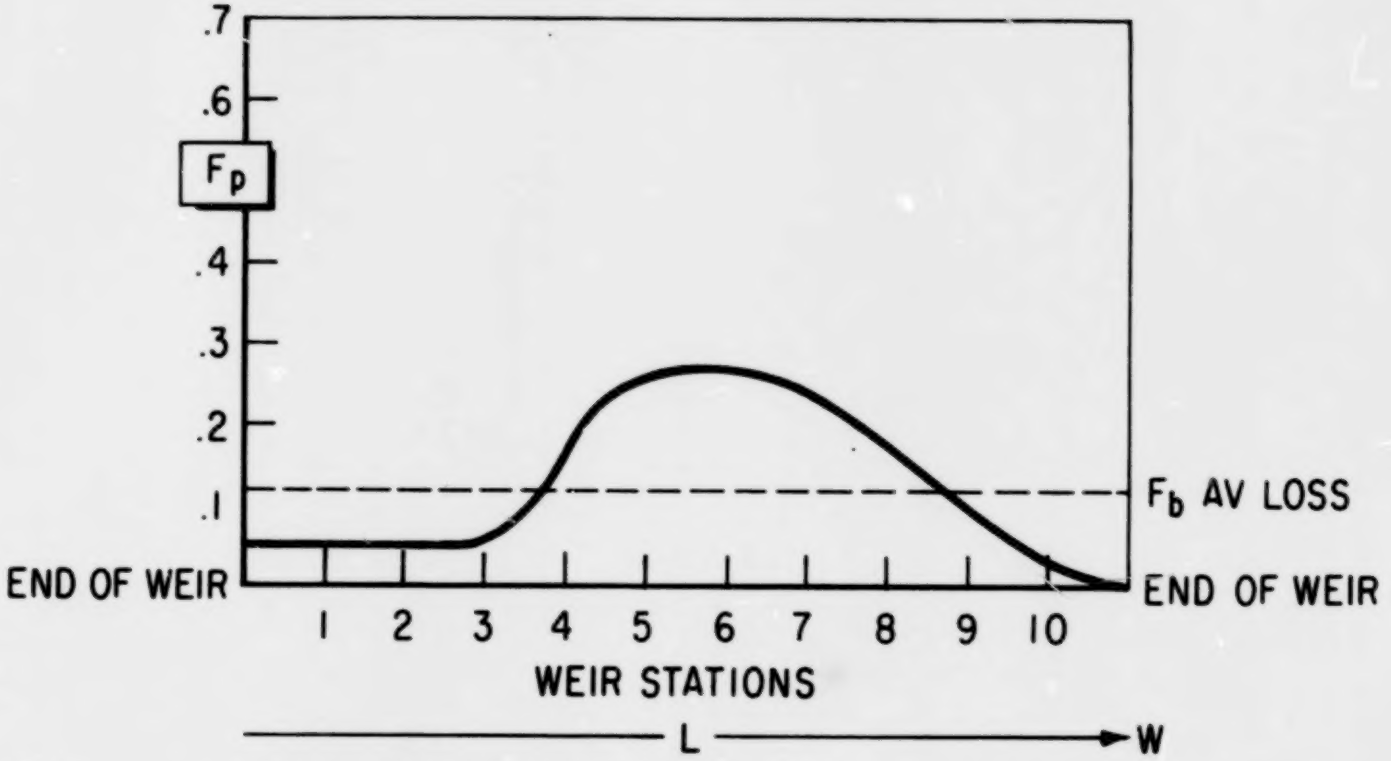
Figure 48 is plotted from a model dye test (Run 31) with all weirs in the basin. Calculations are made from data following the procedure described earlier[2]. The average losses are also given in Figure 48. The predicted loss of suspended solids from the MRPS model and the loss relative to MRPS basin flow are presented in summary, in Figure 49.

PREDICTED PERFORMANCE AT THE MRPS

The area required for removal of fully flocculated Class I solids as a function of overflow rate are shown in Figure 50. Intercepts with the 60,000 sq ft ordinate are the predicted removals for various flows to the existing MRPS basin. An area of 80,000 sq ft would be required for 85 percent removal for a normal event handling 305 cfs (137,000 gpm) combined sewage. Approximately double the area is required for a 10 percent increase in removal from 85 to 95 percent. It can readily be seen that for influents of 102 cfs containing 75 to 250 mg/l suspended solids, the predicted effluent quality would be 15 to 50 mg/l from the existing MRPS. The

Figure 47

DISTRIBUTION OF LOSS ALONG WEIR

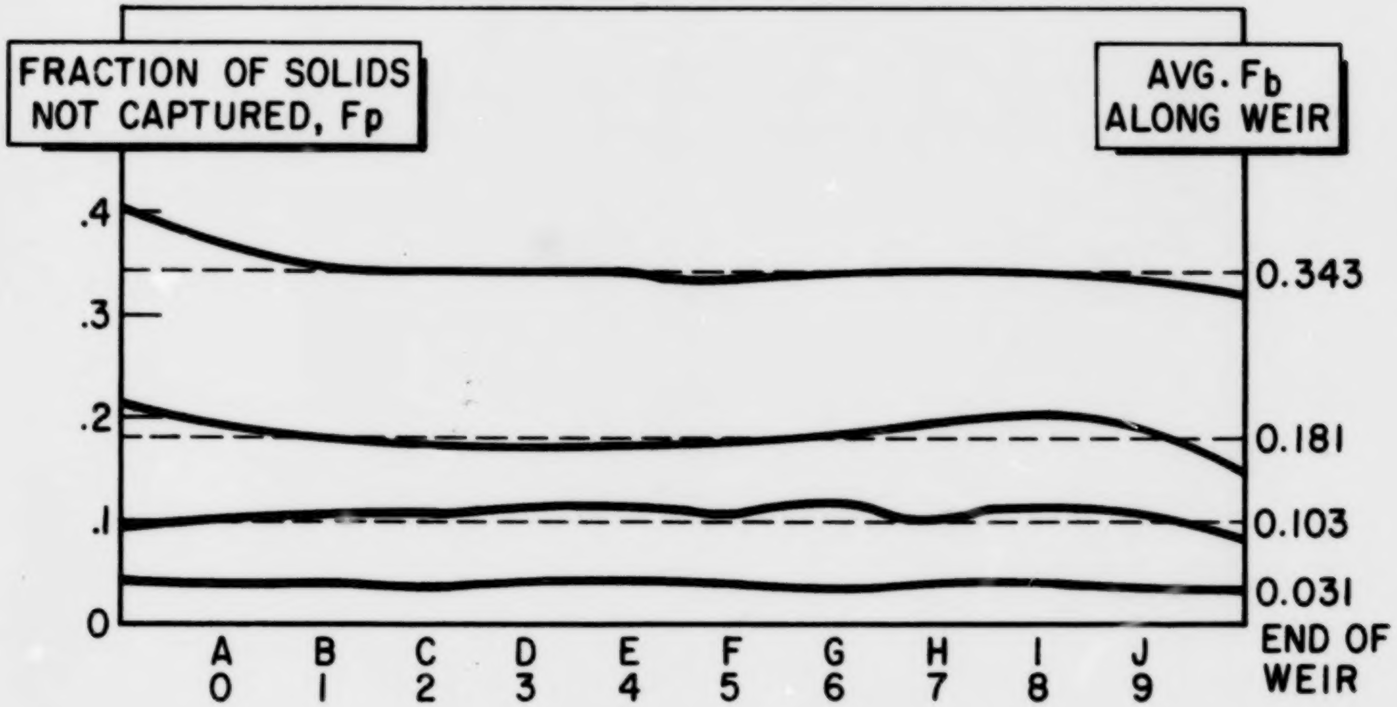


WEIR STATIONS ASSUMED EQUALLY SPACED

AVERAGE ALONG WEIR = $F_b = 0.12$

Figure 48

AVERAGE LOSS ALONG PROTOTYPE EFFLUENT WEIR



	RUN	MODEL (GPM)	PROTOTYPE (CFS)	F_b
a.	*	25	102	.031
b.	38	40.8	166	.103
c.	31	75	305	.181
d.	**	102	410	.343

* RESULTS AVERAGED FROM RUNS 31 & 35
** RESULTS COMPUTED FROM RUN 35

Figure 49

PREDICTED LOSS FROM THE MRPS PROTOTYPE WITH FIVE WEIRS

SYSTEM: 20 mg/l BAROID HECTORITE CLAY
20 mg/l PURIFLOC C31
COMBINED SEWAGE 166 mg/l GRAVIMETRIC SOLIDS

CLASS I SEDIMENTATION OF FULLY FLOCCULATED SOLIDS

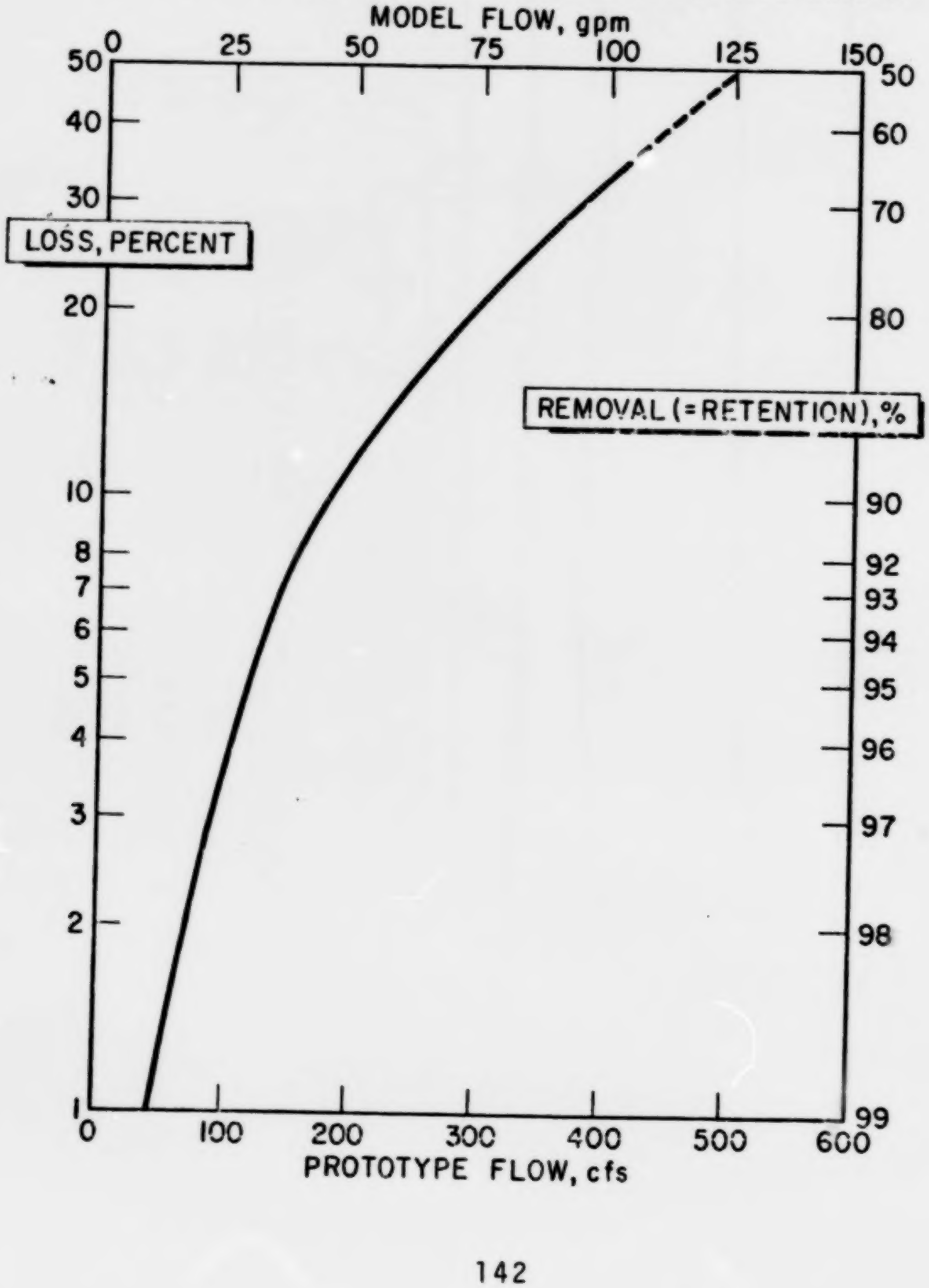
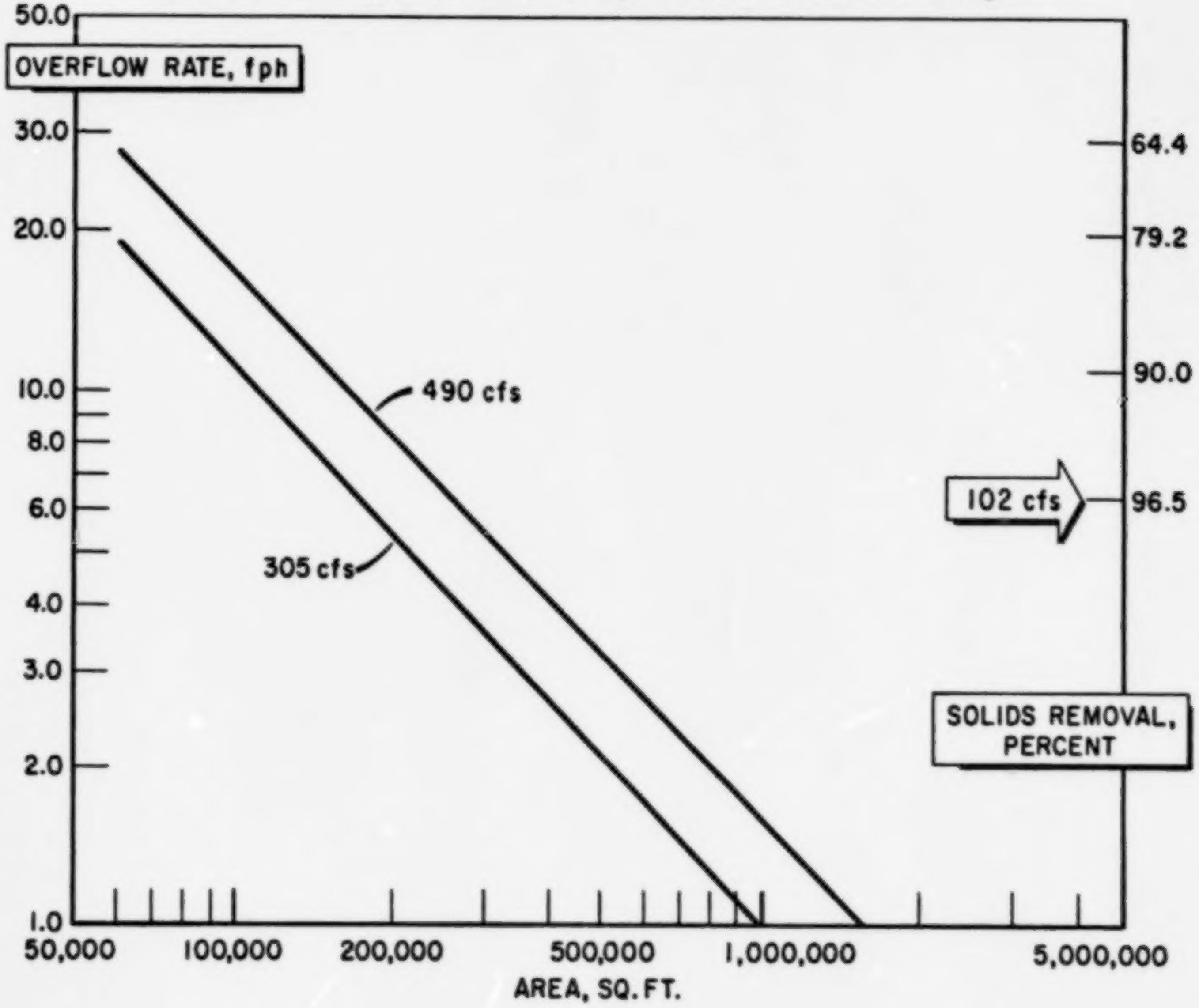


Figure 50

PREDICTED AREA REQUIRED FOR REMOVAL OF CLASS I SOLIDS
FLOCCULATED WITH B.H. CLAY 20mg/l & PURIFLOC C31 20mg/l



MRPS could be modified using flocculants to provide a fully-flocculated influent at 187 cfs and achieve approximately 91 percent removal of suspended solids.

The predicted removal efficiencies are based on realistic basin hydraulic conditions simulated in the model and also more idealized settling conditions in the long-tube sedimentation studies. Additional investigation of sedimentation rates developed under various conditions of long-tube mixing would be required before finalization of basin design.

SECTION 9

DISINFECTION

The evaluation of the effectiveness of chlorine and other disinfection systems in the treatment of combined stormwater is of considerable interest at this time. Current information on the disinfection of combined sewer overflows is sparse[1,2], and the effect of untreated combined sewer overflow on public health is of serious concern. The problem has been recognized only in recent years and now methods must be found to treat this flow effectively and economically. Coupled with the interest in disinfecting combined sewer overflows, there is also considerable interest in improving the safety practices associated with disinfection. The need for safer alternatives for chlorine gas is becoming more apparent as population densities in urban areas around water and wastewater treatment increase. The press of urban and suburban development has made once undesirable land near sewage treatment plants or pumping stations now more acceptable. The situation at Milk River combined sewer overflow pumping station is a case in point. There are potential dangers of maintaining large inventories of chlorine required to adequately treat resultant waste flows. Some larger metropolitan centers such as New York, Chicago, and New Orleans, have already converted to relatively safe, though significantly more expensive sodium hypochlorite[3,4].

Four major problems must be dealt with in the chemical treatment of combined sewer overflows: (1) the tremendous volumes, (2) the relatively short time available for treatment, (3) control of large quantities of potentially dangerous chemicals commonly used for disinfection, and (4) extremely variable rates of flow. The ultimate disinfectant should be safe to handle, relatively efficient, and reasonably economical.

OBJECTIVES AND METHODS

The primary objective of the disinfection work at the Milk River Pumping Station (MRPS) was to evaluate chlorine and other disinfectants for the treatment of combined sewer overflow. In the course of fulfilling this objective, data were also gathered on dry-weather sewage to provide a basis for evaluating treatment efficiencies. A number of parameters considered were: disinfectant dispersion, disinfectant concentration, and contact time.

Dispersion actually encompasses three items: the form of introduction (gas vs. liquid, minor stream vs. total stream), type of diffusion, and degree of mixing. It is also important to consider efficiency as a function of concentration. It is desirable to have a chlorine residual sufficient to continue killing pathogens after treatment has been completed, but not strong enough to endanger desirable aquatic organisms such as fish. Another critical parameter is contact time. Concentration (C) and contact time (t) are related to kill efficiency (E) in the following manner[5,6]:

$$E = K\ t\ C^n$$

where K incorporates certain demand-exerting characteristics of the sewage, such as H_2S or other reducing compounds, and n is a rate constant of the reaction.

All of the disinfectants used in this study, regardless of their source ($BrCl_5$, Cl_2, NaOCl, etc.), immediately hydrolyze to form hypohalous acids in solution. The undissociated hypohalous acid species is the most effective form of halogenated disinfectants according to current theory. In the case of chlorinated systems, it is generally accepted that the neutral HOCl molecule is able to penetrate the cell much more readily than its charged conjugate, reacting with vital enzymes and thereby destroying the cell.

Secondly, the amount of hypochlorous acid formed is dependent on the source and the degree of dissociation, which is sensitive to pH. Hypochlorous acid dissociates in the following manner:

$$HOCl \rightleftharpoons H^+ + OCl^-$$

where[7] $K_a = 3.2 \times 10^{-8}$. The concentration of undissociated hypochlorous acid decreases rapidly with increased pH[8]. The pH of the sanitary sewage flow at the MRPS is typically around 7.4. Combined sewage overflow pH varied from 7.0-7.5. Fielding et al.[9] have pointed out that many investigators today are ignoring the effect of pH. At pH 6.0, 97 percent of the HOCl is present in an undissociated form; at pH 7.4 only 50 percent undissociated HOCl is available.

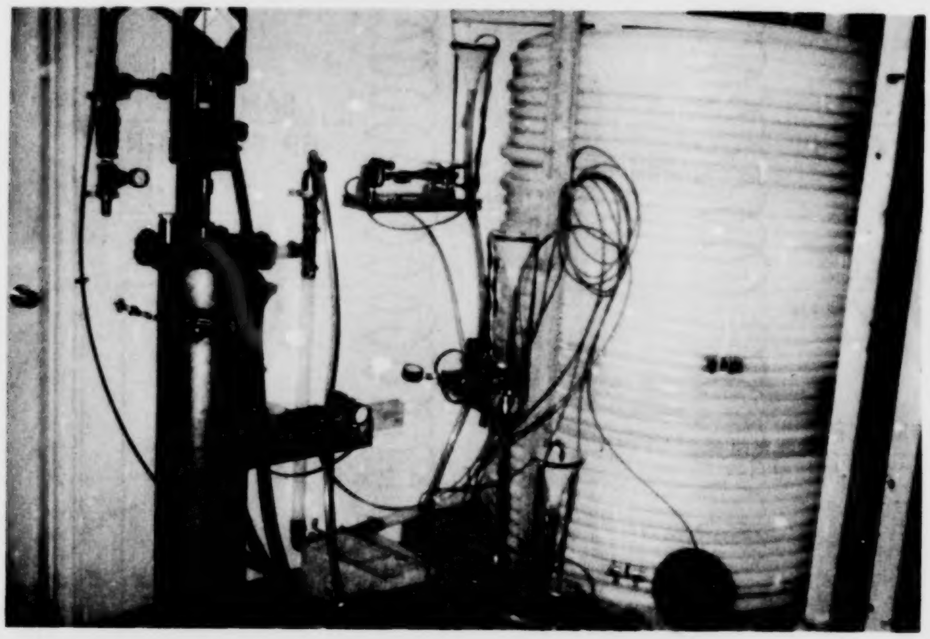
A third factor is the effect of reducing materials such as nitrogenous compounds in the sewage. These convert HOCl to chloramines which are thought to be about 20 percent as effective as HOCl. Conversion is reported to be a maximum at pH 8.3[10]. This phenomenon does not present a problem with bromine compounds; bromamines do form but they seem to be about as effective as HOBr[11].

DISINFECTION PILOT PLANT

Combined sewage overflows are heterogeneous systems which can vary abruptly in chemical and physical character. Control systems and experimental systems measured on different days make comparisons difficult. A parallel pilot plant, using one side as a control and one for experimentation, was designed to avoid this problem. This system also permitted direct comparison of any two disinfectants. A schematic view of this arrangement is shown in Figure 51. The pilot plant flow was pumped from the wet well of the pumping station to a constant head tank which had a detention time of less than two minutes.

Figure 51

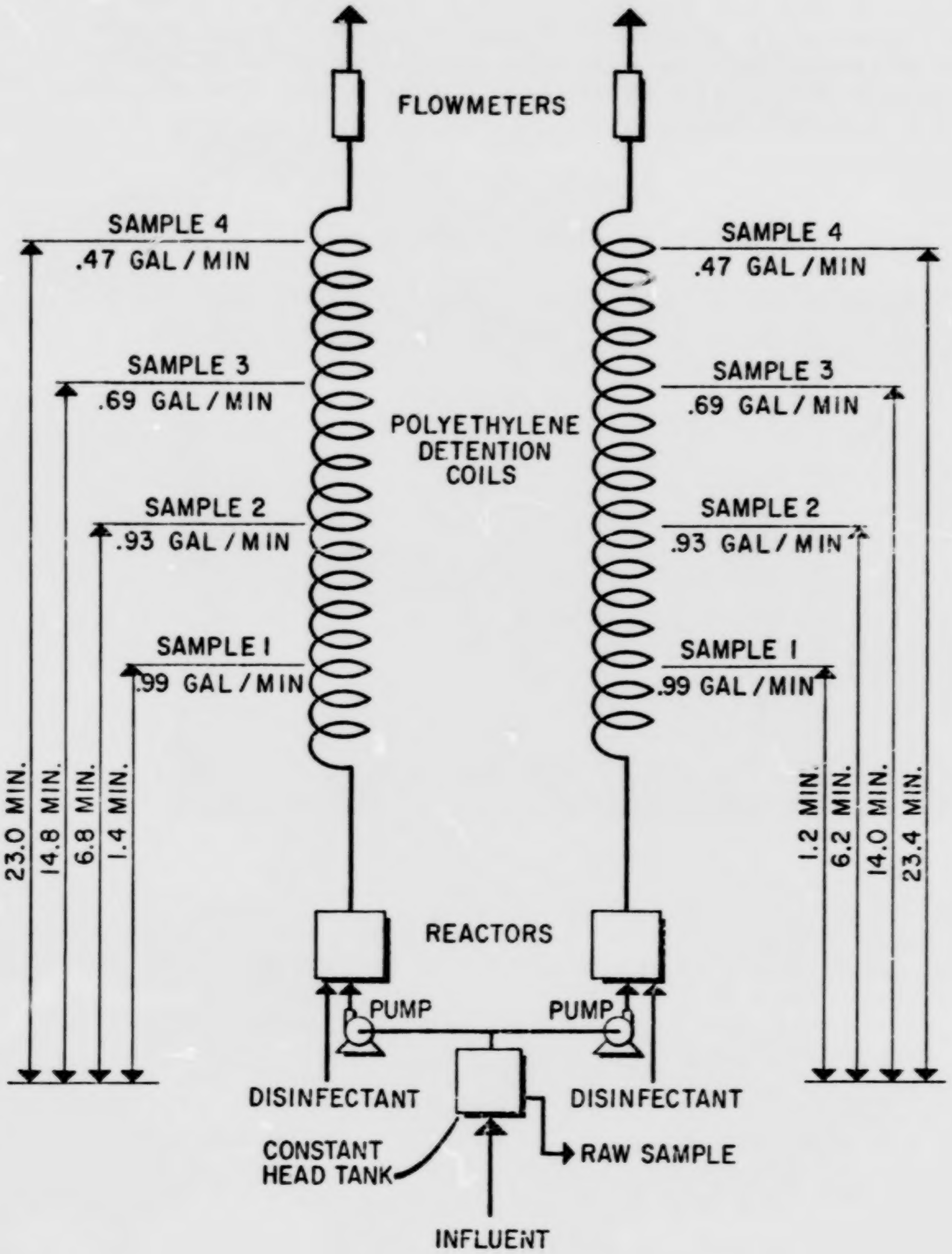
Disinfectant Reaction Chamber and Detention Coil



A variable speed pump serving each plant conducted the flow into a reactor chamber (10" dia., 5' high) fitted with a variable speed stirrer and baffled to increase mixing efficiency (Figure 51). Five disinfectant addition points were provided along the side of the reactor to allow variance of reaction time. The disinfected flow then proceeded to a coil of polyethylene tubing (1.5" ID, 1500' long) which was equipped with four sample points along its length (Figure 52). Detention time in the coil was about 23 minutes as determined by timing the passage of dye fronts between the disinfection injection points and the sampling ports.

Figure 52

SCHEMATIC OF PARALLEL OPERATION OF DISINFECTANT PILOT PLANT


FLOWMETERS
SAMPLE 4
.47 GAL / MIN
SAMPLE 3
.69 GAL / MIN
POLYETHYLENE DETENTION COILS
SAMPLE 2
.93 GAL / MIN
SAMPLE 1
.99 GAL / MIN
23.0 MIN.
14.8 MIN.
6.8 MIN.
1.4 MIN.
SAMPLE 4
.47 GAL / MIN
SAMPLE 3
.69 GAL / MIN
SAMPLE 2
.93 GAL / MIN
SAMPLE 1
.99 GAL / MIN
1.2 MIN.
6.2 MIN.
14.0 MIN.
23.4 MIN.
REACTORS
PUMP
PUMP
DISINFECTANT
DISINFECTANT
CONSTANT HEAD TANK
RAW SAMPLE
INFLUENT

The effluents of both plants were equipped with a flow meter which provided accurate monitoring of the flows through both plants. Effluent flow was maintained at 5 gpm for each plant. Knowing the effluent flow and the flow out each sample point, the total flow through the reactor could be determined and chemical feed rate could be adjusted to maintain the desired feed concentration. The total flow through each reactor was approximately 8 gpm.

These flows and the pilot plant design represented an attempt to achieve 5-25 minutes contact time which was felt to be necessary for adequate disinfection. The flows also had to be of sufficient velocity to prevent undue settling of solids in the contact coil. These considerations then had to be incorporated in the construction of a plant of reasonable size.

Attempts to automate sampling proved unsuccessful; manual sampling was accomplished without difficulty. Two 100 ml samples were collected simultaneously from each plant at the appropriate time and sample point. In those experiments involving variable contact time, the object was to sample the same slug of flow as it proceeded through the plant. The detention times determined from the dye studies allowed this to be done fairly accurately.

DISINFECTANTS AND THEIR APPLICATION

Three disinfection systems were of prime interest: (1) chlorine, (2) sodium hypochlorite, and (3) a chlorine-bromine mixture ($BrCl_5$). A system in which 10 mg/l Br^- was injected ahead of Cl_2 application was also examined.

The various advantages and disadvantages of these systems are summarized in Table XXXIX. Chlorine has a significant economic advantage although it is probably the most hazardous of the systems considered. Recent accidents involving large chlorine gas release in populated areas have caused much concern by public health officials. Chlorine and the other gaseous halogen systems can release harmful gases into the atmosphere during turbulent dispersion. If the concentration is high and applied at a fast rate with accompanying dispersing turbulence, this atmospheric accumulation can be considerable. All of the gaseous systems are highly corrosive.

Because of its common usage, chlorine is useful as a performance standard against which other disinfectants were measured. Sodium hypochlorite has received some attention recently as a replacement for chlorine and warranted examination in this study[3,4]. The NaOCl system is advantageous

TABLE XXXIX

ADVANTAGES AND DISADVANTAGES OF VARIOUS DISINFECTION SYSTEMS

	Advantages	Disadvantages
Chlorine	Low cost Stable	Hazardous Corrosive Gas evolution
NaOCl	Safe Non-corrosive (except to Al) No gas evolution Large quantities easily dispersed	Expensive Higher capital investment Poor storage stability
$BrCl_5$	Fast Stable	Hazardous Corrosive Gas evolution
NaBr + Cl_2	Fast	Hazardous Dual system Corrosive Gas evolution

from a safety standpoint, and can be rapidly dispersed in large quantities. However, it is unstable, expensive (relative to chlorine), and usually requires substantial investment. The NaOCl system is the preferred disinfectant at the Milk River Pumping Station principally because of the high population density in that area. The use of bromine-chlorine mixtures (specifically $BrCl_5$) as disinfectants has not been reported extensively in the literature. Considerable work with these mixtures has been done by The Dow Chemical Company. These systems may be potentially superior to chlorine in efficiency and therefore require reduced volumes to achieve comparable results. While $BrCl_5$ shares many physical properties with chlorine which make it hazardous, a safety advantage might be realized in the reduced volumes required[12].

Chlorine is generally fed as a gas and may be applied either to a minor stream or to the total stream. The pilot plant was set up to feed chlorine from a 15 lb. tank through a minor stream arrangement. The $BrCl_5$ was fed in a similar manner except that it was drawn from its storage cylinder as a liquid. The liquid quickly evaporated in the feed line with the aid of a heat tape and was fed through the minor stream aspirator in the same manner as the chlorine. Liquid $BrCl_5$ was withdrawn to prevent changes in the ratio of bromine to chlorine which would occur if the chemical were withdrawn in the vapor state. Both the chlorine and the $BrCl_5$ systems were controlled by a cylinder mounted chlorinator which was capable of delivering up to 1.5 lb. Cl_2/24 hours.

Sodium hypochlorite was fed via a microfeed pump as a 5.25 percent solution through a small diffuser inserted in the reactor in place of the aspirator. Sodium bromide was fed as a 0.1 N solution in a manner similar to that of NaOCl.

ANALYTICAL PROCEDURES

A number of analyses were performed on samples collected from the pilot plant. These included: chlorine demand[13], chlorine residual, total coliform, fecal coliform, fecal strep, nitrogen[13], total carbon[14], suspended solids[13].

The nature of this work was such that a spectrophotometric technique for total chlorine residual was considerably more convenient than the conventional amperometric technique. To verify the validity of this test and its correlation to the amperometric[13, page 440], a standard curve was run with NaOCl in distilled water. The two chemical delivery systems (NaOCl microfeed pump and the cylinder mounted chlorinator) were then calibrated by both methods by running tap water through the plant. There was good agreement between the methods.

The spectrophotometric technique consisted simply of collecting the sample (about 100 ml) and immediately dosing with about 1-2 grams reagent grade potassium iodide. The intensity of the developed color is proportional to chlorine residual concentration. Measurements were made in less than one minute at 420 nm using a spectrophotometer. Acidification of the sample was not considered necessary prior to analysis for residual chlorine because: (1) pH was not exceptionally high (usually about 7.0) and was not significantly affected by the potassium iodide additions due to the buffer strength of the sewage, (2) the amount of potassium iodide added was far in excess of stoichiometry to insure complete reaction, and (3) suspended solids were not exceptionally high (about

100 mg/l in sanitary flow; 200 mg/l in storm flow) so that significant adsorption of color bodies onto the solids was not a problem.

A second 100 ml sample was also collected at the same time as the residual sample in a bottle containing about 0.5 ml sodium thiosulfate which arrested further disinfection. The samples were then analyzed for total coliform, fecal coliform, and fecal streptococci via the MILLIPORE® technique[15].

EXPERIMENTAL PROGRAM

Initial work was performed using sanitary sewage. Runs were made using chlorine, sodium hypochlorite, and $BrCl_5$, at conditions of 10 mg/l as Cl_2, 100 percent mixing speed, and fed at the lowest disinfection addition point on the reactor. The initial variable was contact time. From this point the work proceeded into two areas: varying the concentration of disinfectant and studying the effect of disinfection on flocculation. This experimental program was essentially duplicated on storm sewage.

A summary of the experimental program for storm and dry-weather flow is shown in Table XL. For the purposes of discussing the data, reference is made again to the schematic layout of the plant as presented in Figure 52.

TABLE XL

SUMMARY OF DISINFECTION EXPERIMENTAL PROGRAM

System	Variable Time Constant Dose	Variable Dose Constant Time
Cl_2 vs Cl_2	S	S
Cl_2 vs NaOCl	S*, C*	S
Cl_2 vs $BrCl_5$	C	C
NaOCl vs $BrCl_5$	S, C	S
NaOCl alone	S, C	C
NaBr + Cl_2 vs NaBr + Cl_2		S
NaBr/NaOCl vs NaOCl	S	
NaBr/NaOCl vs Cl_2	S	S
$BrCl_5$ vs Cl_2 + NaBr	C	

*S = Sanitary sewage

*C = Combined sewage

RESIDUAL AND DEMAND vs. DISINFECTANT DOSE

The use of chlorine residual as a feedback device to control disinfectant dose is a practice carried over from water treatment. Unfortunately, wastewater systems are much more complex and maintenance of a given residual level is difficult at best. This is due primarily to continuous and erratic fluctuations in the concentrations of those substances which directly exert a demand on dissolved chlorine (e.g. H_2S). Frequently, if a residual is measured after 10-30 minutes contact, and it is fluctuating noticeably in that period, elaborate dampening devices must be employed to prevent increasing harmonic oscillation in dose correction. Notwithstanding these considerations, residual is still the best measure of dose efficiency available. From an economic as well as a public health standpoint, a goal in disinfection should be to maintain the lowest residual to give the desired bacterial kill.

A number of experiments were performed in this work in which residual chlorine was monitored at a given contact time (1.3 and 6.4 minutes) while dosage was varied from 2.5 to 15 mg/l Cl_2. The data gathered on combined sewer overflows parallel the sanitary data at the lower doses. Chlorine and $BrCl_5$ are almost equal at high doses (10-15 mg/l Cl_2) in terms of residual while limited NaOCl data indicated possibly slightly lower residuals than that found in sanitary flow.

One of the factors which influences reaction rates and the extent of reaction is reactant concentration. Demand may be influenced therefore by disinfectant dose. The data in Table XLI support this supposition. This is an alternative way of comparing the results shown in Table XL since demand is merely the difference between residual and dose at any given time. This type of consideration is necessary from a design engineering viewpoint to determine inventory capacities. The important point to note in Table XLI is that demand usually does increase with dose.

Another variable which is important to disinfection control is the range of variability of chlorine demand. Analysis of 56 influent samples has shown that the chlorine demand had an average value of 7.7 mg/l and varied from a minimum of 3.5 mg/l to a maximum of 12.2 mg/l for a maximum fluctuation of 8.7 mg/l from storm to storm. For any selected storm, the maximum fluctuation in influent chlorine demand was 6.2 mg/l.

TABLE XLI

DEMAND, RESIDUAL, AND BACTERIAL KILL FOR COMBINED SEWAGE*

Contact Time (Minutes)	Demand at Dose of				Residual at Dose of				Dose to Kill 99.99%		
	2.5	5	10	15	2.5	5	10	15	TC	FC	FS
Cl_2											
0											
1.3	2.0	2.7	4.6	7.0	0.5	2.3	5.6	8.0	8.1	8.7	15.0
6.4	2.3	5.0	6.3	6.1	0.2	0.0	5.8	8.9	6.1	7.5	13.1
14.4											
23.0											
NaOCl											
0							10				
11.3							3.9				
6.4	1.5	2.0	7.2	9.1	1.0	3.0	3.7	5.3	8.0	15.0	10.8
14.4							3.1				
23.0							4.5				
$BrCl_5$											
0							10				
1.3	2.5	3.4	5.0	7.5	0.0	1.6	5.6	7.5	8.8	8.4	6.5
6.4	2.5	5.0	4.8	8.0	0.0	0.0	4.9	7.0	9.3	14.0	12.0
14.4							4.7				
23.0							4.4				

*All demands, residuals, and doses to kill 99.99% of bacteria indicated are in units of mg as Cl_2/l.

TC = Total Coliforms
FC = Fecal Coliforms
FS = Fecal Streptococci

BACTERIAL POPULATION vs. DISINFECTANT DOSE

The principal objective of a disinfecting system is to kill pathogenic organisms. As pointed out previously in this report, the kill efficiency of a given disinfectant system is influenced by disinfectant concentration and contact time. Research by The Dow Chemical Company has shown that brominated systems may be very effective at reasonable doses in comparatively short contact time.

The experiments of the previous section, where contact time was held constant at 6.4 or 1.3 minutes, and dose varied from 2.5 to 15 mg/l Cl_2, were monitored for bacterial concentration. The disinfectant dose required to give 99.99% kill at these contact times was measured. It was found that when short contact time is available (<10 minutes), dosing in the range of 8-15 mg/l as chlorine was required. The more dilute stormwater generally required less chemical to achieve a given percent kill than sanitary flow.

RESIDUAL vs. CONTACT TIME

If it is desirable to keep chemical dosing to low levels for protection of receiving waters or because of high chlorine demands of the receiving water, contact time becomes a very important parameter. This is especially important at Milk River where sludge deposits exert a significant chlorine demand. A number of experiments were conducted in which disinfectant dose was held at a constant level (arbitrarily 10 mg/l as Cl_2) and a slug of flow was sampled as it progressed through the plant. Samples were collected and treated in the manner previously described. All systems, with the exception of NaOCl and $BrCl_5$ in sanitary flow, left residuals in the range of 5-6 mg/l as Cl_2. There was no appreciable decrease in residual over the times monitored (Table XLI) indicating that most of the chlorine demand is satisfied within 1.5 minutes. Similar results were obtained for sanitary flow.

BACTERIAL SURVIVAL vs. CONTACT TIME

The experiments described in the previous section were also monitored for changes in bacterial numbers. It is important to know the amount of contact time required for various systems to achieve a certain kill at a given dose. Data of this type are summarized from experiments on storm flow in Table XLII. The average times necessary for various systems to kill 99.99% of total coliform are shown.

TABLE XLII

TIME REQUIRED TO KILL 99.99% OF TOTAL COLIFORMS IN COMBINED SEWAGE*

Run #	$BrCl_5$	Cl_2	NaOCl	Initial Total Coliform	Initial Demand mg Cl_2/l
C-12	4.4	-	4.5	19×10^6	11.3
C-11	-	4.8	5.0	10×10^6	10.2
C-9	1.3	7.4	-	7.6×10^6	10.2
C-7	8.8	8.1	-	8.2×10^6	9.5
C-5	-	-	8.5	4.5×10^6	10.7
C-3	-	-	17.7	13.8×10^6	7.7
C-2	-	-	21.0	6.8×10^6	6.2
C-1	-	-	14.7	8.5×10^6	5.7

*All values in units of minutes using concentrations equivalent to 10 mg as Cl_2/l.

No significant differences in the time required to achieve a 99.99% kill were observed. The data obtained during run C-9 showed a significant decrease in time of kill for $BrCl_5$ over Cl_2 but this was not supported by the other data. Considerably more work would be needed to quantitate any real differences.

EFFECT OF DISINFECTANTS ON CHEMICAL FLOCCULATION

Because of the short time available to treat stormwater, it may be necessary to disinfect simultaneously with other chemical treatment such as flocculation. Jar test experiments were conducted on sanitary flow to determine the effect of disinfection on flocculation by polyelectrolytes.

Both a cationic (PURIFLOC C31) and an anionic (PURIFLOC A23) flocculant system were tested. Using the optimum flocculant dose determined from jar testing, NaOCl was added just prior to polymer addition at variable concentrations or intervening mixing times. After flocculation and settling were completed, the percent optical solids removal from the overhead was determined from turbidimetric measurements. This was taken as an indication of flocculation activity. The removal of optical solids (percent) was calculated using the following formula:

$$(1 - A/A_0) \times 100 = \% \text{ Removal of Optical Solids}$$

where A is the absorbance of the experimental sample and A_o the absorbance of the raw sample. The experiments were carried out in series of liter quantities using a gang stirrer.

In the cationic system the optimum dose of PURIFLOC C31 was 40 mg/l and NaOCl was first added in concentrations from 0 to 80 mg/l as Cl_2 with one minute intervening rapid mix (100 rpm). No impairment to flocculation was observed until 40 mg/l as Cl_2 was exceeded as shown in Figure 53. The NaOCl dose was then held constant at 10 mg/l as Cl_2 and the intervening mixing time between halogen and polymer addition varied between 0 and 5 minutes. No effect on flocculation was observed.

The anionic system usually requires a multivalent cation, in this instance 25 mg/l Fe^{3+}, to promote flocculation. The iron was added, stirred 5 minutes at 80 rpm, followed by NaOCl in concentrations of 0-80 mg/l as Cl_2. After one minute rapid mix, the polymer was added. Flocculation was not affected even up to 80 mg/l as Cl_2. In the next experiment, NaOCl was held at 15 mg/l as Cl_2 and the intervening mix time varied. Again, flocculation was unaffected.

It was concluded that at doses of disinfectant (<40 mg/l Cl_2) chemical flocculation can be carried out simultaneously without impaired performance.

LONG-TERM RESIDUAL OF $BrCl_5$ IN COMBINED SEWAGE

A study was made to compare the long-term effect of Cl_2 and $BrCl_5$ residuals in combined sewage. Samples of combined sewage were taken from each of the two pilot plants after treatment with 10 mg/l $BrCl_5$ and Cl_2, respectively. The samples were held for a period of 24 hours and the residuals checked. The results of this study are shown in Table XLIII.

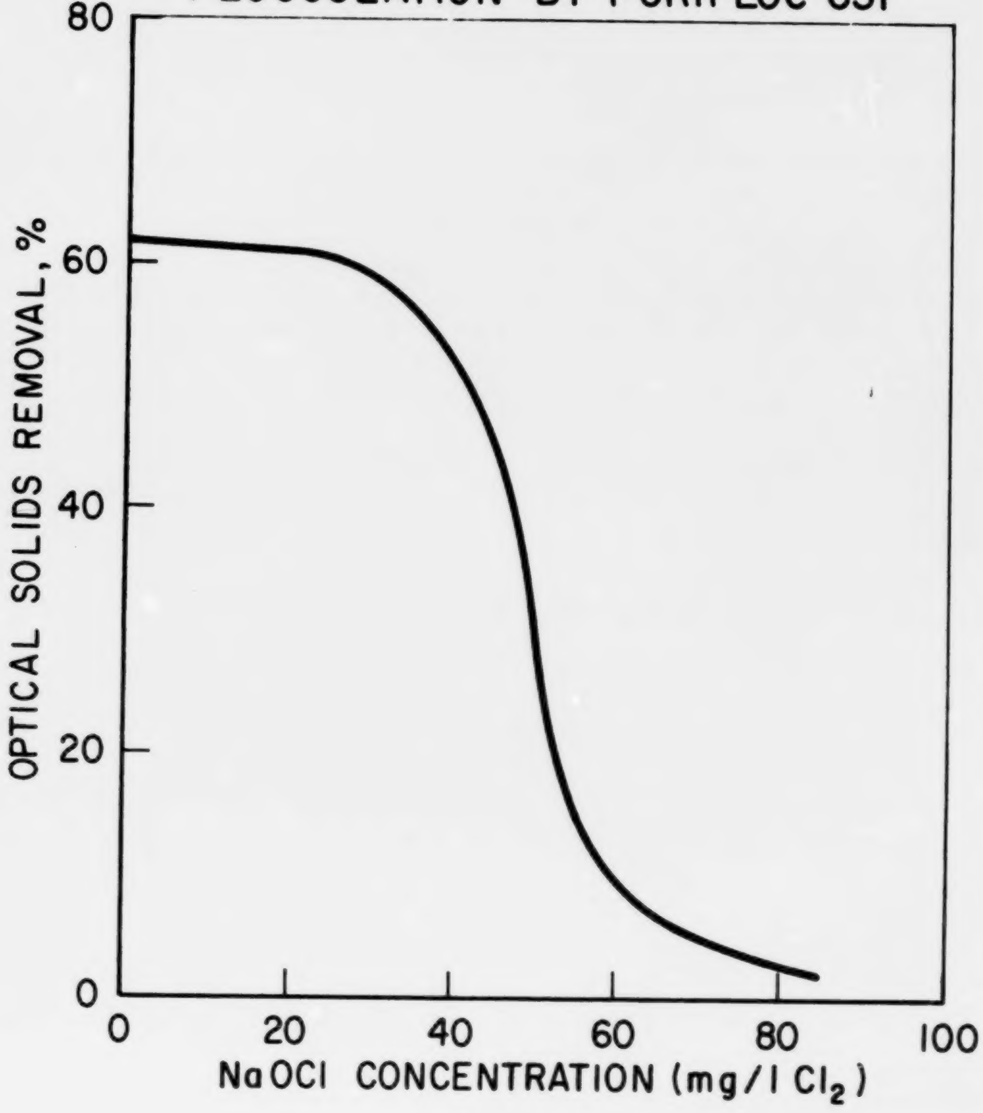
TABLE XLIII

LONG-TERM RESIDUALS OF DISINFECTANTS IN COMBINED SEWAGE

Time (Hours)	Residuals in mg/l Cl_2	 $BrCl_5$
1	5.7	5.1
4	4.6	3.8
20	2.3	2.3

Figure 53

EFFECT OF SODIUM HYPOCHLORITE ON FLOCCULATION BY PURIFLOC C31



Sunlight effects were minimal. Temperature of the samples increased from the temperature of the flowing sewage to approximately room temperature. The samples were not mixed so the effect of turbulence was minimal. As expected, $BrCl_5$ maintained a residual similar to that of the Cl_2 during the 24 hour period.

CHLORINE DEMAND OF THE MILK RIVER

The Milk River is a channel about one mile in length, connecting the MRPS to Lake Saint Clair. It receives almost no dry-weather flow. An ideal condition would be to maintain a slight (0.5-1.0 mg/l Cl_2) residual in the channel but the highly reducing environment precludes this. Anaerobic decomposition of the bottom sludges causes considerable generation of methane, H_2S and NH_3.

In order to assess the conditions in the channel with respect to chlorine demand and bacterial population, two samples were collected at each of the five stations shown in Figure 54. The first sample was taken about one foot below the surface (in the approximate middle of the channel); the second sample was bottom sludge. Five and thirty minute demands were determined for both samples; bacterial population was determined for the overhead sample.

The bottom sludges had extremely high demands. The profile in chlorine demand through the channel of the overhead sample is shown in Figure 55. The demand declined as the channel outlet was approached. The magnitude of the demands by the overheads and bottom sludges demonstrates the impracticability of maintaining a residual in the channel. Disinfection of stormwater will have to occur in the settling basin since the extreme chlorine demand will consume any residual as soon as the flow enters the channel. The populations of various bacterial forms at each station are shown in the histogram of Figure 56. Populations of all three classes were relatively constant except in the clean water of the lake.

DISINFECTION OF MILK RIVER COMBINED SEWER EFFLUENT

Based on theoretical detention times in the existing settling basin, 99.99% of the initial bacterial population could be killed at the MRPS for 95 percent of the overflow periods. An adequate supply of stable disinfectant would be assumed available. The pumping rate was also assumed constant for a given storm. In reality, intense storms of short duration would require increased pumping rates and result in shorter

Figure 54

Stations in the Milk River Channel Sampled for Chlorine Demand and Bacterial Population

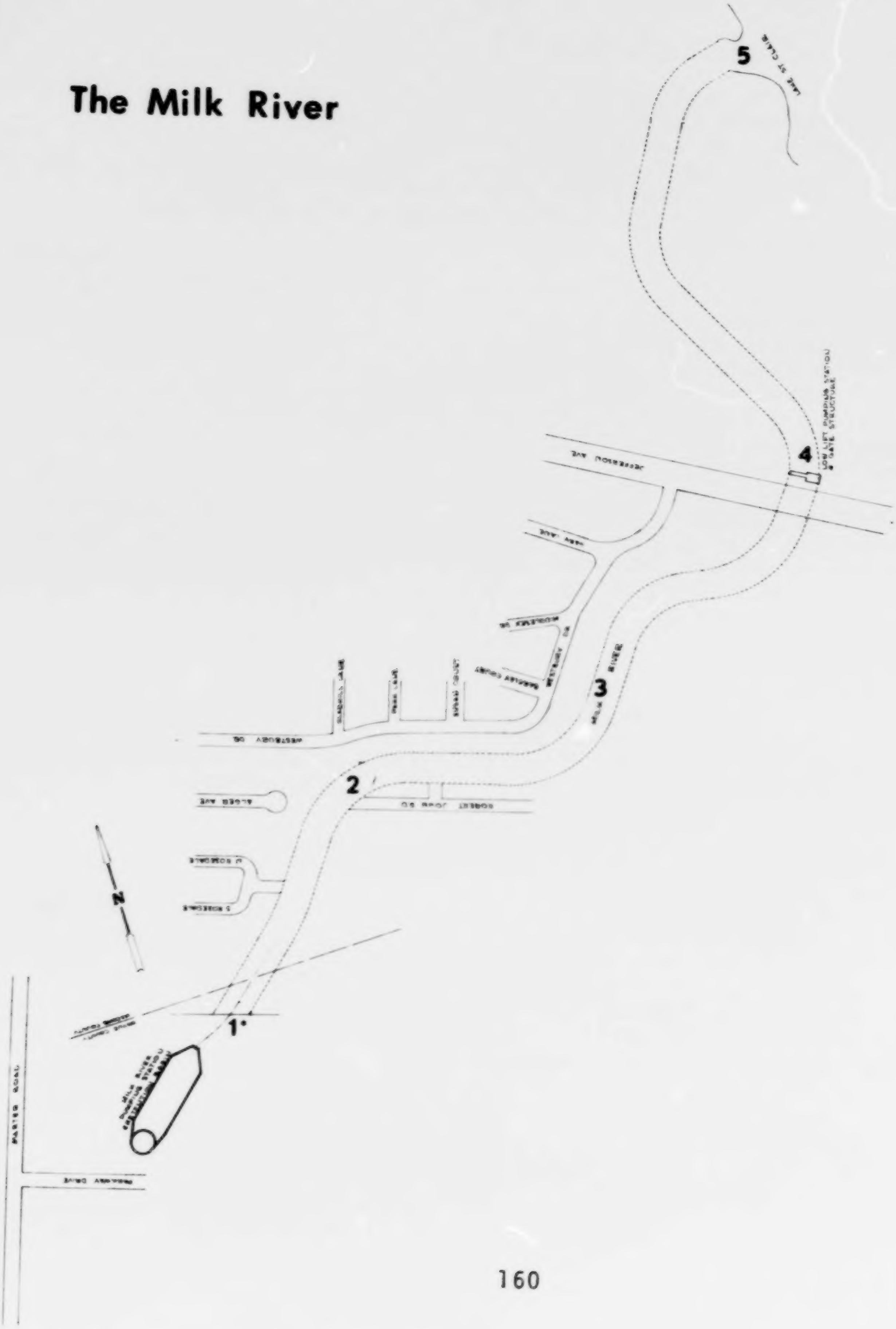

The Milk River
5
4
3
2
1*
N

Figure 55

PROFILE OF CHLORINE DEMAND IN THE MILK RIVER CHANNEL

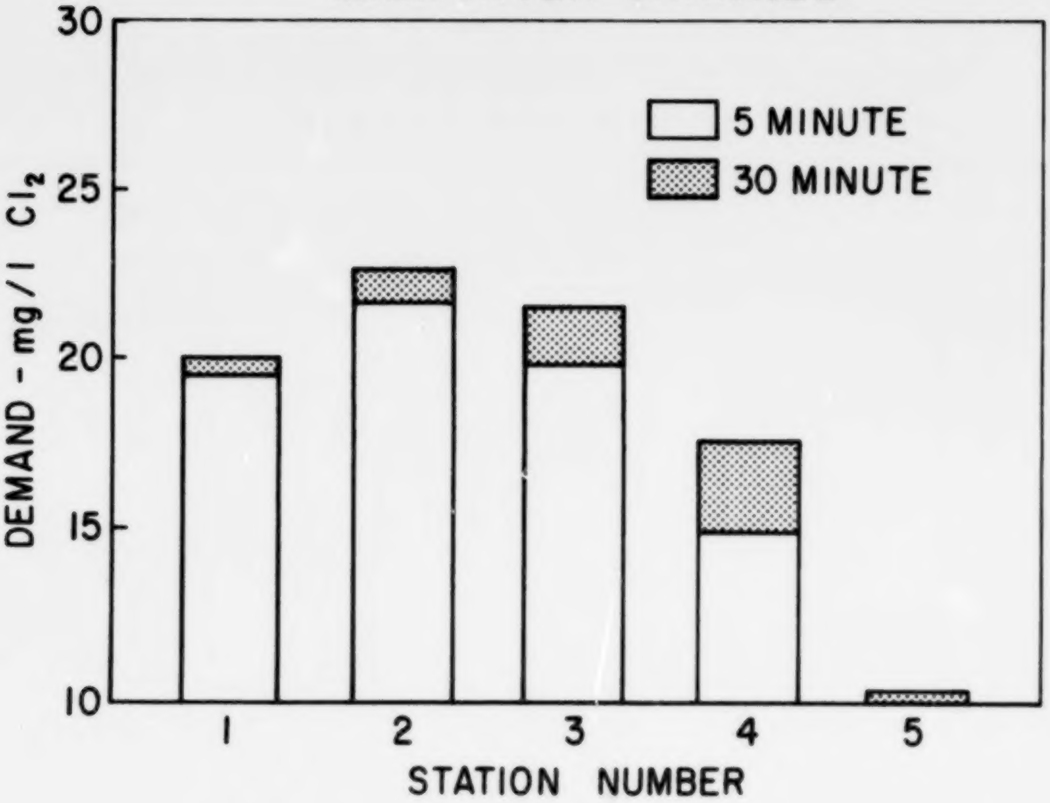
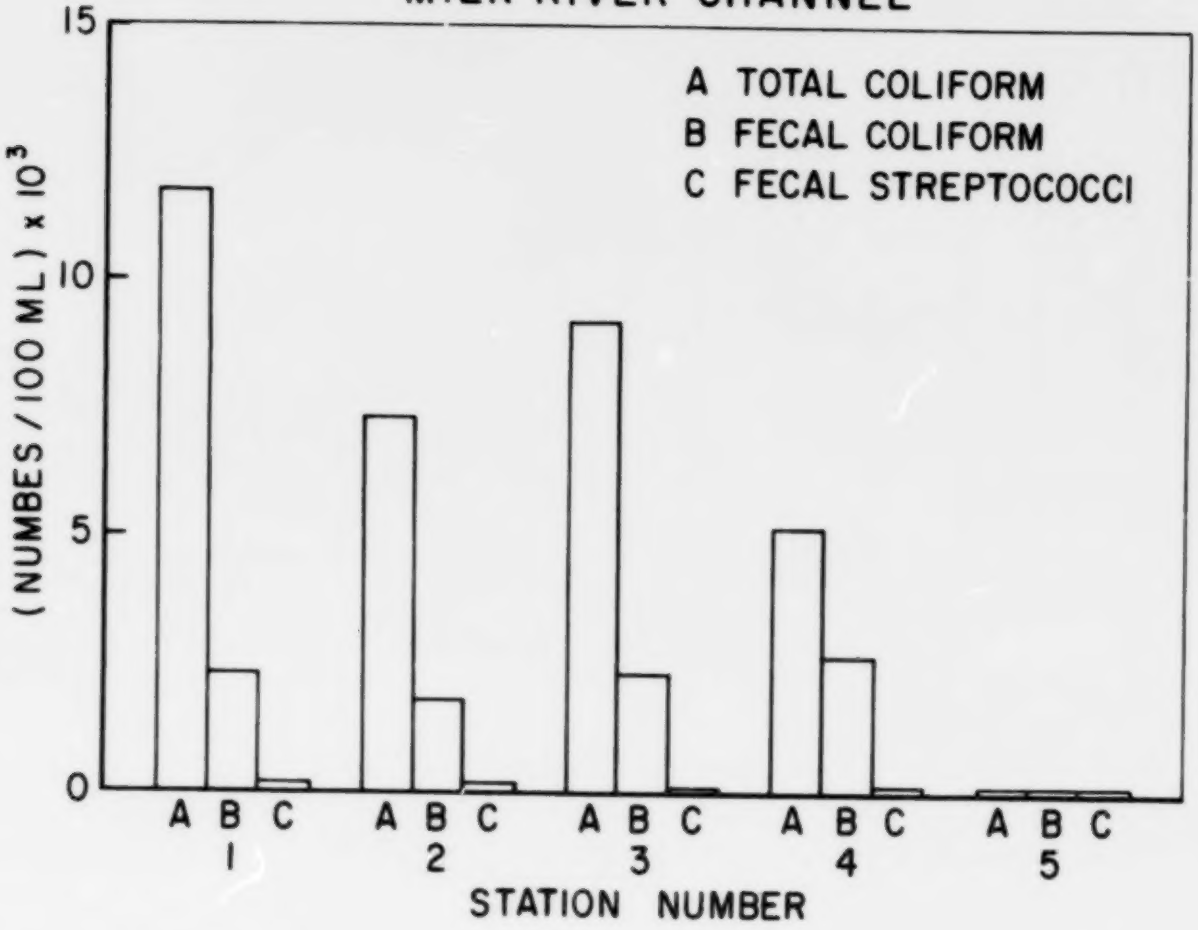


Figure 56

PROFILE OF BACTERIAL POPULATION IN THE MILK RIVER CHANNEL



contact periods in the channel and higher disinfectant concentrations would be required. The frequency of pumping is less and the channel is full toward the end of a given storm. A new demand value must be considered in such cases. Accurate dosing during a storm event is a statistical problem of huge proportions.

A statistical analysis of the frequency of volumes pumped for all storm events occurring at the MRPS over an eight year period reveals that 95 percent of pumping time involves volumes less than 8 million cubic feet (59.8 x 10^6 gal). Further, 95 percent of the time the pumping rate is less than 715 cfs (321,000 gpm). Since the basin volume is .508 million cubic feet (3.8 x 10^6 gal), the theoretical contact time will be at least 11.8 minutes. Even with detention inefficiencies, it appears that this is sufficient time to achieve 99.99% kills at a dose of 10 mg/l as Cl_2 with any of the systems.

Because of extensive short-circuiting in the basin, modifications of the hydraulic design would be required to approach the theoretical detention times. The disinfectant then could be added at the pump discharge. Adequate dispersion would be available, contact time would be sufficient, and simultaneous flocculation would not be adversely affected.

CORRELATION DATA

During the course of the work at the MRPS, considerable data were collected on influent storm flow, e.g. various forms of suspended solids, bacterial counts, chlorine demand, etc. The correlations between several of these variables were determined via computer. The main objective was to discover a statistically significant ($\geq$0.70) correlation between chlorine demand and some easily or at least immediately measured parameter. The disadvantages of the present system of gauging chlorine demand by delayed residual information has been previously discussed.

Correlations between chlorine demand and other parameters were extremely poor. Total coliform did not correlate to the solids data. Fecal coliform displayed the highest correlations with the solids and nitrogen data, but no relation to time of storm duration.

It was concluded that correlation of disinfection variables with easily measured parameters at the MRPS was not possible.

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SECTION 10

ECONOMIC ANALYSES

GENERAL

Numerous factors affect the ultimate cost of combined sewer overflow treatment systems. Analysis of the frequency distribution of flow rates for the Milk River Pumping Station (MRPS) suggests a rapidly decreasing economic benefit if overflow treatment capacity is increased. Some measure of this relationship is established in the following analysis of the MRPS data. The analysis is valid for many combined sewer treatment systems where the following characteristics prevail.

1. Capital costs vary approximately in proportion to plant capacity.
2. The occurrence of flow rates follows a logrithmic normal distribution similar to that at the MRPS.

Data presented in Table XLIV are derived from the frequency distribution analysis shown in Table IX. A flow of 2450 cfs was assumed to represent the peak plant capacity for a ten-year storm (100%). The percentage of peak flow for each of the rates shown was then calculated. Benefits were assumed to be the percent of total volume treated; treatment of all volumes equaled 100 percent.

TABLE XLIV

PROPORTIONAL COSTS AND BENEFITS AT VARIOUS FLOW RATES

Rate* (cfs)	Cost % Peak Rate	Benefit % Volume Treated
305	12	45
410	17	74
610	25	78
915	37	85
1220	50	89
1535	63	94
1840	75	97
2155	88	97
2450	100	100

*1 cfs = 448.8 gpm

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It can be seen from Table XLIV that the rate of benefit decreases significantly when plant size reaches approximately 20 percent of peak flow, suggesting that an economic breakpoint in combined sewage treatment facilities may occur at about that point. A final breakpoint would have to be developed from cost analyses of specific alternate plans. The foregoing relationship, however, provides a rational beginning for economic evaluation of the Milk River combined sewer overflow treatment problem.

FLOCCULATION AND SEDIMENTATION

Unlike the design of most conventional sewage treatment plants, designs of combined sewer overflow treatment systems must consider the logarithmic normal distribution of flow rates. Accurate estimates of treatment efficiency and cost-benefit relationships cannot be derived without this consideration. A plant capacity once established must be examined for efficiency over the expected range of flows. These flows must be weighted according to their probable frequency of occurrence.

A condensation of the contributing effects of variable rates and volumes of flow at the MRPS to the degree of solids removed by the PURIFLOC C31-BAROID HECTORITE clay system is contained in Table XLV. The lowest flow rate is that for which the existing retention basin could operate with about 96 percent solids removal efficiency; the highest flow rate is that which is near the economic breakpoint for capital investment. Note that the 50 percent rate occurs near the lower limit.

The flow volumes were selected to cover a range overlapping the 50 percent volume occurrence. It should be noted that the relative magnitudes of storm volumes can influence the predicted removals. If the volume retained by the existing basin is large relative to the volume overflowed into the Milk River, then the actual removals may be greater due to a longer effective detention time. As the volume retained in the basin becomes a smaller fraction of the total volume treated, then the removals should approach the predicted removals.

The selected rates of overflow, including the statistical mean (50 percent occurrence) tabulated in the first column and the corresponding percentages of occurrence included in the second column were taken from Figure 12. The predicted percentages of solids removal in the third column were taken from Figure 50. If an equal solids removal of 85 percent is desired, then the required net areas for the selected overflow rates as taken from Figure 13 are included in the fourth column.

TABLE XLV

RELATIVE SOLIDS REMOVAL AS A FUNCTION OF OVERFLOW RATE AND VOLUME

Overflow Rate (cfs)	Occurrence[1] ≥Overflow Rate (%)	Predicted[2] Solids Removed in Existing Basin (%)	Net Area to Achieve 85% Solids Removal (1000 sq ft)	Predicted Solids Removed[4] (fraction[6], tons) in Existing Basin at Treated Volumes of				Notes
				Million cu ft .512 % Occurrence[3] ≥ 35	1.2 50	2 80	4 90	
102	25	96	60[5]	52[6]	121	202	404	One-third small pump
				2.5[7]	6.0	9.9	19.9	rate.
187	50	91	60	43	100[8]	167	333	Mean rate.
				2.4	5.7	9.4	18.9	
305	70	80	80	33	77	128	256	One small pump rate.
				2.1	5.0	8.3	16.6	
410	80	65	110	24	56	93	187	One large pump rate.
				1.7	4.0	6.7	13.5	
490	90	52	130	19	45	76	151	20% of 2450 cfs,
				1.4	3.2	5.4	10.8	i.e., 10-year design flow.
				↑ Figures this column apply to equivalent volume of basin	↑ Figures this column apply to mean volume of basin			

1 Taken from Figure 12.
2 Taken from Figure 50.
3 Taken from Figure 13.
4 Percentages assuming 50% rate occurrence - 50% volume occurrence[6] as 100%; actual solids removed (T) based on average solids concentration of 166 mg/l.
5 Net area of existing basin
6 Percentage of 50% rate occurrence - 50% volume occurrence
7 Tons of solids removed.
8 50% rate occurrence - 50% volume occurrence.

Several selected volumes of overflow and the corresponding percentages of occurrence as taken from Figure 13 head the remaining tabulations. Two tabulated predictions of solids removal are contained in Table XLV for each selected rate and volume of overflow. The upper value is the relative solids removed on a weight basis as a percentage based on a 100 percent value for the 50 percent rate - 50 percent volume occurrence. The lower value is the actual weight (tons) of solids predicted to be removed. This analysis is based upon actual detention times determined from the hydraulic studies.

The effects of variable design flow rates, variable storm volumes, and settling basin areas on suspended solids removal are included in the foregoing analysis.

Several systems appear worthy of further investigation. They are:

1. Modified MRPS System - This system involves replacement of one 305 cfs pump with a variable system, installation of two transverse baffles, construction of a flocculation system designed for 85 percent removal of suspended solids at a rate of 102 cfs, and disinfection.

2. New Plant - This system includes pumping, construction of a flocculation system with a settling basin designed for 85 percent removal of suspended solids at a rate of 490 cfs, and disinfection.

The data used in calculating the net removal efficiency for each of the two systems described above are given in Tables XLVI and XLVII. Steady-state conditions are assumed for each generalized class interval. This factor must be taken into account in the final design.

DISINFECTION

Selection of a disinfection system for installations such as the MRPS requires careful consideration of several important factors.

Firstly, a decision must be made whether to use gas phase systems, e.g. Cl_2 or $BrCl_5$, or a liquid phase system involving NaOCl. Considerable discussion in the literature has concerned the economic merits of Cl_2 vs. NaOCl.

The conversion from the use of Cl_2 gas to solid NaOCl by the city of New York was discussed by Steffensen and Nash[1] who concluded: "the total annual costs using liquid chlorine or sodium hypochlorite are substantially equal. The use of sodium hypochlorite appears desirable as the difficulties

TABLE XLVI

PREDICTED REMOVAL FROM MODIFIED EXISTING BASIN

Overflow Rate (cfs)	% Removal	% Volume at Rate Shown	% Volume x Mean Storm Volume (10^6 cu ft)	Total Solids at Rate Shown (lbs)	Predicted Solids Removed at Rate Shown (lbs)
102	96	0.08	0.096	994	954
187	91	.14	.168	1740	1583
305	80	.20	.240	2486	1989
410	65	.18	.216	2237	1454
490	52	.15	.180	1864	969
>490	30	.25	.300	3107	932
		1.00	1.200	12430	7881

TABLE XLVII

PREDICTED REMOVALS FROM OPTIMIZED BASIN

Overflow Rate (cfs)	% Removal	% Volume at Rate Shown	% Volume x Mean Storm Volume (10^6 cu ft)	Total Solids at Rate Shown (lbs)	Predicted Solids Removed at Rate Shown (lbs)
102	96	0.08	0.096	994	954
187	95	.14	.168	1740	1653
305	93	.20	.240	2486	2312
410	91	.18	.216	2237	2036
490	85	.15	.180	1864	1584
>490	50	.25	.300	3107	1553
		1.00	1.20	12430	10092

and hazards involved in the delivery and handling of the materials are considerably less." Baker[2] stated that in such comparisons only the cost of a chlorinator vs. a diaphragm pump is considered and the cost of the entire feed system (provision for storage, etc.) which for the NaOCl system can be considerable. He concludes that, "Differences in chemical costs favor the selection of liquid chlorine in calculating operational costs, and more than offset capital expenditure differences in nearly every case when capital costs are calculated for the entire chemical feed system."

The proximity of inhabited structures to most combined sewage overflow installations, such as the MRPS, safety dictates use of the safer NaOCl system for disinfection.

The extent of disinfection to be provided must also be selected. A kill of 99.99 percent of total coliforms can be accomplished successfully within the residence time required for flocculation of the combined sewage at the MRPS without measurable detriment to flocculation or clarification of the overflow. A kill of 99.99 percent was arbitrarily chosen to provide an effluent concentration averaging about 1000 total coliforms per 100 ml. This level is comparable to standard levels established in many recreational areas.

The chlorine demand in the Milk River channel has previously been shown to be extremely high. This condition prevents utilization of the channel for disinfection by residual chlorine in the plant effluent even though sufficient contact time is available before discharge into Lake Saint Clair. Dilution of plant effluents in large streams or lakes has a similar effect in reducing the effectiveness of chlorine residuals in sewage plant effluents.

From these considerations and the results of the disinfection studies, satisfaction of the average chlorine demand of the influent combined sewage at the MRPS might be the best level to attempt disinfection. The chlorine demand did not vary greatly within a single storm or between distinctly separate storms. Disinfection with NaOCl at a level of approximately 10 mg as Cl_2/l might effect satisfactory total coliform levels without attendant dangers of residual chlorine occurring during periods of low chlorine demand.

The final decision to be made is the selection of flows for optimum treatment. The distribution of flow rates at the MRPS produces a rapidly diminishing degree of treatment per dollar of invested capital beyond flow rates approximately equal to 20 percent of the ten-year design flow (490 cfs).

If chlorine feed facilities were provided for an overflow rate of 490 cfs, approximately 75 percent of the total volume pumped from the MRPS would contain total coliform levels of about 1000 per 100 ml or less. The additional 25 percent of the volume would also be treated with NaOCl up to the capacity of the 490 cfs system. Effluent coliform counts in this fraction, however, would vary upward from 1000 per 100 ml, depending on the rate of flow and the resulting concentration of disinfectant.

Disinfection with NaOCl involves five major cost elements: land, storage facilities, chemical feed systems including pumps, chemical costs, and operation and maintenance costs. Fixed costs are due to land, storage facilities, and the chemical feed system; costs of chemicals, and operation and maintenance vary with the degree of treatment. Land costs are not considered in this report since they vary considerably with plant location.

The volume of 7.5 percent NaOCl solution required for treatment of volumes of overflow shown, at a concentration equivalent to 10 mg/l as Cl_2 is shown in Table XLVIII. Commercial bleach (trade %) is diluted 1:1 to provide 7.5 trade percent available chlorine. This is necessary to maintain maximum solution effectiveness over long periods. Dilution can be made with potable water in the tank while providing air sparging or other mixing techniques.

TABLE XLVIII

7.5% NaOCl REQUIRED TO TREAT VARIOUS VOLUMES AT 10 mg/l

Volume Treated (10^6 cu ft)	7.5 Trade % Available Cl_2 Required (gal)	% Occurrence
3	1572	75
6	2360	14
9	3146	6
12	3932	3
15	7864	1

It may be most economical to provide underground storage for approximately 10,000 gallons of 7.5 percent NaOCl solution. Sufficient disinfectant to treat several large storms and the capability to treat five to six storm volumes of more probable occurrence would be provided. Replenishment of storage would be required about every one to two months based on average yearly storm frequencies.

The disinfection system should be designed to treat storm overflow rates of from 100 to 490 cfs with NaOCl at a level of 10 mg as Cl_2/l. This would require a chemical feed system capable of pumping and dispersing a 7.5 percent NaOCl solution at rates from 6 to 30 gpm. Two all-titanium or all-Teflon centrifugal feed pumps capable of delivering from 6 to 20 gpm appear reasonable when considering flexibility and standby protection.

An estimate of the installed costs to provide disinfection for the Milk River Combined Sewer Overflow according to the system described above is included in Table IL.

TABLE IL

FIXED COSTS OF DISINFECTION

10,000 gal lined storage tank	$ 7,500	
Two chemical feed pumps	6,000	
Controls, meters, piping, and diffusers	3,000	
Total Installation Cost		$16,500
Overhead, profit, and contingencies (25%)		4,125
Total Cost		$20,625

Assuming a life of 20 years and an interest rate of 8%, the annual cost for disinfection equipment would be $1,796. Average yearly storm pumping is approximately 767 million gallons making the fixed costs for disinfection approximately $0.002/1000 gallons. Assuming an average cost of NaOCl of $0.012/pound of available Cl_2, the total yearly cost for NaOCl would be about $7,700 or $0.010/1000 gallons.

Treatment of 75 percent of the flows at the MRPS with NaOCl to provide an average total coliform concentration of about 1000/l would cost approximately $0.012/1000 gallons. Installation cost of disinfection equipment would be approximately $20,625; chemical costs, less operation and maintenance, would be approximately $7,700 annually.

SUMMARY

A cost summary showing the fixed and operating costs associated with the existing station, the proposed modified station and a new plant, is contained in Table L.

TABLE L

SUMMARY OF COST ANALYSIS OF MILK RIVER TREATMENT SYSTEMS

	Design Rate (cfs)	Net SS Removed (%)	Present Installation Costs		Flocculation		Disinfection		Total Costs[1,4]	
			Fixed	Operating	Fixed	Operating	Fixed	Operating	Fixed	Operating
Existing Plant	-	-	4,190,000[3]	185,000	-	-	20,625	7,700	4,210,625	192,700
Modified Plant[1,2]	102	63	-	-	256,320	190,000	20,625	7,700	4,466,945	382,700
New Plant	490	81	-	-	3,775,000	341,000	20,625	7,700	7,327,625	533,700

[1] Includes present cost of Milk River Pumping Station less cost of the retention basin.

[2] Assumed that sludge can be discharged by gravity to the Detroit interceptor.

[3] Corrected to 1969 dollars by ENR Index Factor (1.73)
Pumping Station 3,532,000
Retention Basin 658,000

[4] Does not include cost of additional sludge flushing water.

SECTION 11

ACKNOWLEDGMENTS

While it is impossible to acknowledge all agencies and organizations who have contributed to the success of the Milk River Project, special mention must be made of the following:

PARTICIPATING AGENCIES

Milk River Drainage Board

Wayne County Drainage Commission

SUBCONTRACTORS

Dorr-Oliver, Inc. - Stamford, Connecticut

Pate, Hirn and Bogue, Inc. - Detroit, Michigan

ORGANIZATIONS PROVIDING TECHNICAL ASSISTANCE

City of Grosse Pointe Woods, Michigan

Detroit Edison Company

ESSA, Office of the State Climatoligist - Lansing, Michigan

Department of Interior - FWQA, Detroit Program Office

Grosse Ile, Michigan

Southeast Michigan Council of Governments

U. S. Coast Guard - Detroit, Michigan

Grateful acknowledgment must also be extended to numerous individuals, too many to list here, but without whose help this work could never have been completed.

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SECTION 13

GLOSSARY OF TERMS AND ABBREVIATIONS

GENERAL

°C - degrees Centigrade
cfs - cubic feet per second
cu ft - cubic feet
ENR - Engineering News Record
ESSA - Environmental Sciences Service Administration
°F - degrees Fahrenheit
fph - feet per hour
fps - feet per second
ft - feet
FWQA - Federal Water Quality Administration
gal - gallons
gpd - gallons per day
gpm - gallons per minute
in - inches
LD_{50} - lethal dose, 50% mortality
mg/l - milligrams per liter
ml - milliliters
ml/l - milliliters per liter
min - minutes
MRPS - Milk River Pumping Station
nm - nanometer
rpm - revolutions per minute
sec - seconds
SEMCOG - Southeastern Michigan Council of Governments
sq ft - square feet
USGS - United States Geographical Survey

BACKGROUND DATA AND ANALYSIS

BOD - Biochemical Oxygen Demand
CA - calcium
CL - chloride or chlorine
COLI - coliform
DEM - demand
FEC - fecal
HARD - hardness
INR - inorganic
N - nitrogen
ORG - organic
P - phosphorus
SET - settled
SMPL - sample
SOL - solids
STRM - storm
STRP - Streptococci
SUS - suspended
TEM - temperature
TOT - total
TUR - turbidity
VOL - volatile or volume

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LONG-TUBE SEDIMENTATION STUDIES

ADEPL - actual depletion (ft/spl)
DBOT - corrected depth from bottom (ft)
DF - final depth of suspension in LTSD (in)
D(M) - uncorrected depth from bottom (ft)
DO - initial depth of suspension in LTSD (in)
DSUR - corrected depth from surface (ft)
GS - gravimetric solids
GSO - initial average gravimetric solids (mg/l)
L - length dimension
LDSUR - $\log_{10}$ DSUR
LTSD - long-tube sedimentation device
LTSPL - $\log_{10}$ TSPL
M - depth sampled (1,...,P)
N - time sampled (1,...,Q)
OS - optical solids (absorbance x 1000)
OSO - initial average optical solids (absorbance x 1000)
P - total depths sampled
Q - total times sampled
RGS - removal of gravimetric solids
ROS - removal of optical solids
SETR - settling rate (DSUR/TSPL) (ft/min)
T - time dimension
TDEPL - theoretical depletion (ft/spl)
T(N) - uncorrected time (min)
TSP - sampling period at each depth (min)
TSPL - corrected time (1/10 min)

HYDRAULIC ANALYSES AND MODEL STUDIES

C - concentration of suspended solids
C_o - concentration of suspended solids in feed
F - fraction of suspended solids not captured at selected value of h/t
F_b - fraction of suspended solids not captured in basin
F_p - fraction of suspended solids not captured in local basin
h - distance below surface in batch test
H - depth of continuous settling basin
(Use volume/surface area for basin for varying depth)
L - length along basin discharge weir
N_{Re} - Reynolds number
N_{Fr} - Froude number
t - settling time in batch test
W - Width of weir
V - characteristic velocity of basin (arbitrarily chosen to be the overflow rate)
ρ - density of fluid
μ - viscosity of fluid

DISINFECTION

A - absorbance (white light)
A_o - initial absorbance (white light)
c - concentration of disinfectant
E - kill efficiency
K - empirical constant
K_a - dissociation constant
n - rate constant
t - contact time

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11020	---	12/67	Problems of Combined Sewer Facilities and Overflows, 1967, (WP-20-11)
11023	---	05/68	Feasibility of a Stabilization-Retention Basin in Lake Erie at Cleveland, Ohio
11031	---	08/68	The Beneficial Use of Storm Water
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11000	---	01/70	Storm and Combined Sewer Demonstration Projects - January 1970
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